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THE DEVELOPMENT AND APPLICATION OF A STANDARD COMPACTION-DEGRADATION TEST FOR SHALES

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Interim Report

THE DEVELOPMENT AND APPLICATION OF A STANDARD COMPACTION-DEGRADATION TEST FOR SHALES

TO:	H. L. Michael, Director Joint Highway Research Project	October 23, 1979
		Project: C-36-5L
FROM:	C. W. Lovell, Research Engineer Joint Highway Research Project	File: 6-6-12

Attached is an Interim Report on the HPR Part II Study titled "Design and Construction Guidelines for Shale Embankments". This is Interim Report No. 7 and is titled "The Development and Application of a Standard Compaction-Degradation Test for Shales". It is authored by Barney C. Hale of our staff.

The research developed a standard test which can be used to predict both the compaction characteristics and mechanical degradability of a shale prior to its use in an embankment. It is shown that the standard AASHTO impact test is suitable, and that the index of crushing so generated discriminates among the relative degradabilities of Indiana shales.

The Report is submitted for review, comment, and acceptance as partial fulfillment of the objectives of the referenced HPR study.

Respectfully submitted,

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Interim Report

THE DEVELOPMENT AND APPLICATION OF A STANDARD COMPACTION-DEGRADATION TEST FOR SHALES

by

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In cooperation with the

Indiana State Highway Commission

and the

U.S. Department of Transportation
Federal Highway Administration

The contents of this report reflect the views of the author who is responsible for the facts and the accuracy of the data presented herein. The contents do not necessarily reflect the official views or policies of the Federal Highway Administration. This report does not constitute a standard, specification, or regulation.

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16. Abstract

Non-durable but mechanically hard shales pose a particular problem in embankment construction. These shales should ordinarily be degraded and compacted in relatively thin "soil" lifts. However, this may be difficult to accomplish because of the shale hardness. The engineer obviously needs to anticipate the difficulty of degrading the shale during construction, and this research has produced a simple standard test for such prediction. The standard AASHTO impact type test can be used to generate compaction relationships and degradability measures simultaneously. The shale gradation before and after compaction is used to generate an index of crushing. Degradation varies directly with this index.

It may ultimately be possible to use the more simple point load strength test to accomplish the prediction. This report shows good correlation between the secant modulus in the point load strength test and the index of crushing.

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LIST OF ABBREVIATIONS AND SYMBOLS

ASTM	American Society for Testing and Materials
IC	Index of Crushing
ISHC	Indiana State Highway Commission
PLS	Point load strength
10-FACT	10% Fines Aggregate Crushing Test
C_u	Coefficient of uniformity
D_{10}	Aggregate size corresponding to 10% passing on cumulative distribution curve
D_{60}	Aggregate size corresponding to 60% passing on cumulative distribution curve
P/D^2	Point load strength index where P is the failure load and D is the sample thickness
Å	Angstroms
R^2	Coefficient of multiple determination
ln	Natural logarithm
mi	mile
ft	feet
in	inch
lb	pound
min	minute
pcf	pounds per cubic foot
psi	pounds per square inch
mm	millimeter

cm	centimeter
m	meter
km	kilometer
kg	kilogram
kN	kilonewton
kPa	kilopascal
MPa	Megapascal

HIGHLIGHT SUMMARY

The poor performance record of compacted shale embankments has led to the current practice of thoroughly degrading non-durable shales for placement as soil fill in embankments. The special problems presented by non-durable but mechanically hard shales has increased the importance of understanding shale degradation during compaction. The development of laboratory degradation tests could be useful in establishing compaction and degradation functions which may ultimately be related to field compaction.

Impact and static tests were performed on three non-durable Indiana shales in this study. The results from these tests led to the development of a single standard test procedure based on the impact compaction method. The standard impact test was then used to analyze the effects of initial gradation, maximum aggregate size, and moisture on shale degradation and compacted density. Correlations between the results of point load strength tests and the degradation values from the standard impact test were also established.

1. INTRODUCTION

The excessive settlements and failures of many embankments constructed of shale materials led to major investigations concerning the properties and behavior of shales. A major slope failure within a shale and limestone embankment along I-74 in Dearborn County, Indiana led to the initiation of an extensive shale research program by the Indiana State Highway Commission through the Joint Highway Research Project at Purdue University. Other investigations into compacted shale embankments were undertaken by the U.S. Army Corps of Engineers Waterways Experiment Station. The reports agree that the deterioration of the shale due to weathering plays a major role in the poor performance record of the embankments.

Durable shales, which can withstand the weathering processes, will perform satisfactorily when placed as rock fill. Non-durable shales however, must be thoroughly broken down during compaction and placed as soil fill. Shales which are mechanically hard but non-durable present special problems relative to construction techniques.

Current practices of breaking non-durable shales down into soil fill places additional importance on understanding shale degradation during compaction. Laboratory tests may be helpful in defining the compaction and degradation functions of shales. These functions may ultimately be related to field conditions.

The work by Bailey (1976) established a basis for laboratory degradation tests. This study concentrated on the development of a single standard testing procedure and its application to troublesome Indiana shales.

In this study, impact and static compaction procedures were evaluated. The impact test was selected as the standard compaction-degradation test and was used to analyze the effects of initial sample gradation, maximum aggregate size, and moisture on shale degradation and compacted density. The standard impact test was also considered along with the point load strength test as a method of predicting shale degradability.

2. REVIEW OF SHALES AS EMBANKMENT MATERIALS

2.1 Definition and Occurrence of Shales

Shales are the most abundant of the common sedimentary materials. Although shales are generally defined as argillaceous sediments which display fissility, a large number of definitions have been developed (Deo, 1972). The definition presented by Pettijohn (1957) and Underwood (1967) is that shale is the more highly indurated and generally fissile equivalent of claystone and/or siltstone. For the purposes of this study, the definition was expanded to include claystone and/or siltstone so that representative hard non-durable samples, exhibiting the desired physical properties, could be selected for laboratory testing and study.

Shales and other sedimentary deposits are widely distributed over the surface of the earth's crust. Shales in Indiana range in age from the Ordovician to the Pennsylvanian periods and dominate the bedrock geology (Austin, 1975). A thick layer of glacial till overlying the bedrock often prevents the exposure of the shales. As the thickness of the till layer decreases in the southern portions of Indiana, shale outcroppings become more frequent and may be encountered during normal construction operations. Further discussion of the shale formations in Indiana are presented by Harrison and Murray (1964) and Austin (1975).

2.2 Significant Engineering Properties of Shales

The basic definition provided by Pettijohn (1957) and Underwood (1967) includes shale materials with a wide range of physical properties. Mead (1936) proposed a classification system which divided

shales into two groups; compaction and cemented shales. The compaction shales are consolidated by the weight of overlying sediments and lack significant amounts of intergranular cementation. The cemented shales are strongly bonded by either cementing agents or by recrystallization of the clay minerals. The compaction shales are generally softer and more subject to slaking (a rapid disintegration caused by cycles of wetting and drying) than the cemented shales. The cemented shales are harder, more durable, and may be successfully used as rock-like materials in embankment construction.

According to Pettijohn (1957), the fissility exhibited by shales is the result of both the compaction and concomitant recrystallization during formation, and the parallel orientation of the micaceous constituents at the time of deposition. Ingram (1953) used three dominant types of breaking characteristics to classify the fissility of shale into massive, flaggy, or flaky categories. Massive shales have no preferred direction of breaking and produce blocky fragments. Flaggy shales break into fragments of varying thickness which have much greater lengths and widths. The pieces have two approximately parallel flat sides. The flaky shales split along irregular surfaces parallel to the bedding planes and produce flakes, thin chips, and wedge-like fragments.

In addition to fissility a shale may be laminated. The laminations may be attributed to: (1) alterations of coarse and fine particles, (2) alterations of light and dark layers based on organic content, (3) alterations of calcium carbonate and silt (Pettijohn, 1957).

The compressive strength of shales may vary from 172 kPa (25 psi) to 103400 kPa (15,000 psi) (Underwood, 1967). Although the compaction shales are generally weaker, they may still display substantial strength.

Indiana shales can not be described in one general statement. The physical properties and mineralogies may vary not only among the different shale units of the various formations, but very often also within a local area, both vertically and laterally within a single formation. The variation in properties is directly related to the environments of deposition and the provenance (source areas of the sediments which made up the shales) of the shale unit.

The younger shales of the Pennsylvanian and upper Mississippian Systems tend to be basically non-homogeneous materials. The shales of the lower Mississippian and older Systems tend to retain more similar physical characteristics throughout the formations.

Shales may also exhibit swelling when exposed to moisture. Shales having clay fractions composed primarily of montmorillonite and illite will swell more than shales with kaolinite and chlorite.

The properties presented above are especially important in the construction of shale embankments. These and other properties are discussed in detail by Pettijohn (1957) and Underwood (1967).

2.3 Problems Related to Compacted Shale Embankments

Road cuts for highways constructed in southern Indiana often encounter shale. Economic and environmental considerations generally make the use of the excavated material in nearby compacted embankment sections more desirable. However, the poor strength and durability characteristics of many shales, along with inadequate construction procedures, have resulted in several undesirable experiences in compacted shale embankments.

Excessive settlement and slope failures of large shale embankments have occurred in several states (Chapman and Wood, 1974). A major slope failure within an embankment on I-74 in Dearborn County, Indiana occurred in December, 1971 and January, 1972 (Wood, Lovell and

Deo, 1973). The embankment had been constructed as a rock fill using a mixture of limestone and shale.

The embankment failures led to the initiation of research and development programs by the Indiana State Highway Commission through the Joint Highway Research Project at Purdue University (Wood, Sisiliano, and Lovell, 1978). Reports from these studies have been completed by Deo (1972) and Chapman (1975) on the classification of shales, Bailey (1976) on shale compaction and degradation characteristics, van Zyl (1977) on the storage and retrieval of existing data on Indiana shales, Abeyesekera (1978) on the shear strength parameters of compacted shales, and Witsman (1979) on the compressibility of compacted shales.

In addition to the work at Purdue, the U. S. Army Corps of Engineer Waterways Experiment Station (1975) conducted a three phase shale research project for the Federal Highway Administration. Phase I of the project was devoted to a survey of the problem areas and current practices. Phase II concentrated on the evaluation and remedial treatment of shale embankments. Phase III developed detailed guidelines for the sampling and evaluation of shales and procedures for the design and construction of shale embankments.

The basic problems with Indiana shales as embankment materials result from either the fissility or non-durability of the shale. Flaggy or flaky shales may break into slabs or platey fragments during excavation and compaction. The pieces may then bridge or arch over large spaces creating a series of irregularly occurring voids within the embankment. Changes in the material properties or loading

conditions during the service life of the embankment will cause collapse of the openings resulting in substantial internal movements (Wood, Lovell, and Deo, 1973).

The non-durability of shale is presently considered the major underlying cause of the embankment problems (U.S. Army Corps of Engineers Waterways Experiment Station, 1975). The slaking of shales resulting from the wetting and drying cycles leads to the deterioration of shale pieces, reduction in material strength, and blockage of drainage paths. According to Abeyesekera (1978), the changes in the physical properties of shales due to weathering will affect the strength properties of the entire mass.

In previous construction practices, shales which appeared competent were placed in large pieces as rock fills. Softer shales were placed as soil fill, but the presence of harder sedimentary rocks (limestone or sandstone) prevented complete compaction (Wood, Lovell, and Deo, 1973).

These procedures left large voids within the embankments. Disintegration of the shale led to collapses of the openings, a loss of interlocking among the pieces, and disruption of drainage resulting in serious settlements and possible embankment failures.

According to Deo (1972) shales which are identified as non-durable or "soil-like" should be thoroughly broken down during construction to eliminate the presence of large voids within the compacted mass. This approach is supported by Wood, Lovell, and Deo (1973) and is presently used by the Indiana State Highway Commission.

Soft, non-durable shales generally do not present a major problem for break down during compaction. However, many shales in Indiana are hard and difficult to degrade despite their lack of durability. For these shales, special compaction procedures must be used to increase the probability of a successful embankment service life.

2.4 The Importance of Shale Degradation Tests

The policy of thoroughly breaking down shales during embankment construction emphasizes the importance of understanding shale degradation resulting from compaction. Appropriate definition of the degradation functions would be most directly achieved through field compaction tests. Yet the expense of field tests and the limitations of present knowledge on shale degradation would reduce the effectiveness of any major field testing program. A standard compaction-degradation test could be used to generate the degradation functions in the laboratory using an assortment of compaction variables. Ultimately, the results and experiences from the laboratory studies could be coupled with field compaction observations. With sufficient data from both laboratory and field studies, the quantitative prediction of shale degradation during field compaction could be made from the results of laboratory testing only.

The work by Bailey (1976) established a basis for shale degradation studies. He performed four types of laboratory compaction tests on samples of Attica shale and analyzed the relationship between degradation, compactive effort, and unit weight. From these results, Bailey found that both densification and degradation appeared to be self-limiting reactions regardless of compactive effort, moisture

content, or initial gradation. Bailey selected the static compaction test from the four laboratory methods as the most appropriate procedure for a compaction-degradation index test.

In addition to the compaction tests, Bailey (1976) investigated the possible uses of the Scleroscope hardness test and point load strength test as indices for the engineering properties of shales. He also performed long term soaking tests to assess shale degradation caused by slaking.

The results reported by Bailey (1976) encouraged continued study in the area of shale degradation and are reflected throughout this report. Using Bailey's experiences as guidelines, this study was devoted to the development of a single standard test procedure and the application of the test to selected, troublesome Indiana shales. In addition to the compaction tests, correlations between the point load strength test and degradation results were attempted.

3. DEGRADATION TEST PROGRAM

3.1 Guidelines of the Testing Program and Selection of Variables

The development of a testing program leading to a standard degradation index test required the consideration of a basic set of guidelines. The list of criteria given below provided guidelines either (a) essential to any test or (b) especially suitable for the testing of shales.

1. The test results must be replicable within tolerable limits.
2. Equipment for the test should be available in typical geotechnical laboratories.
3. The test must induce degradation in the sample.
4. The test must differentiate among various shales.
5. The test should be easily performed and accomplished in a short period of time.

A testing process which could produce repeatable results and could be performed in a typical geotechnical laboratory would be assured if the above guidelines were followed. The criteria to induce degradation and differentiate among shales provided minimum limits for compactive effort and eliminated very severe degradation tests. Finally, the criteria for a simple and quick test favored a process which could test a large number of samples; an important characteristic due to the variability in shale materials.

The degradation of a shale that occurs during compaction is affected by a number of factors. The most evident factors are: the type of shale, the method of compaction, and the compactive effort. Other variables include, the initial gradation, maximum size, and moisture content.

To reflect the influence of these variables, the testing program included three different shales, two methods of compaction, and a range of compactive efforts. The initial gradation, maximum size, and moisture content were held constant for each shale as a standard testing procedure evolved. The effects of these variables were studied later in the testing program.

3.2 Numerical Representation of Gradations

During the compaction process, fracturing, abrasion, and moisture effects break down individual shale pieces. The result is a compacted material with a gradation different from that of the uncompacted material. Therefore, a measure of the gradation change serves as an indicator of the amount of degradation that has occurred.

Several methods may be used to represent the grain size distribution curve when plotted as percent finer by weight vs. a logarithmic scale of grain size diameter. Figure 1 shows the use of the grain size distribution curve in representing the initial and final gradations of a compacted sample. However, graphical methods are not suitable for numerical treatment and can become cluttered when a large number of samples must be compared.

Gradation coefficients have been developed to provide a numerical value representing a grain size curve. The "effective size"

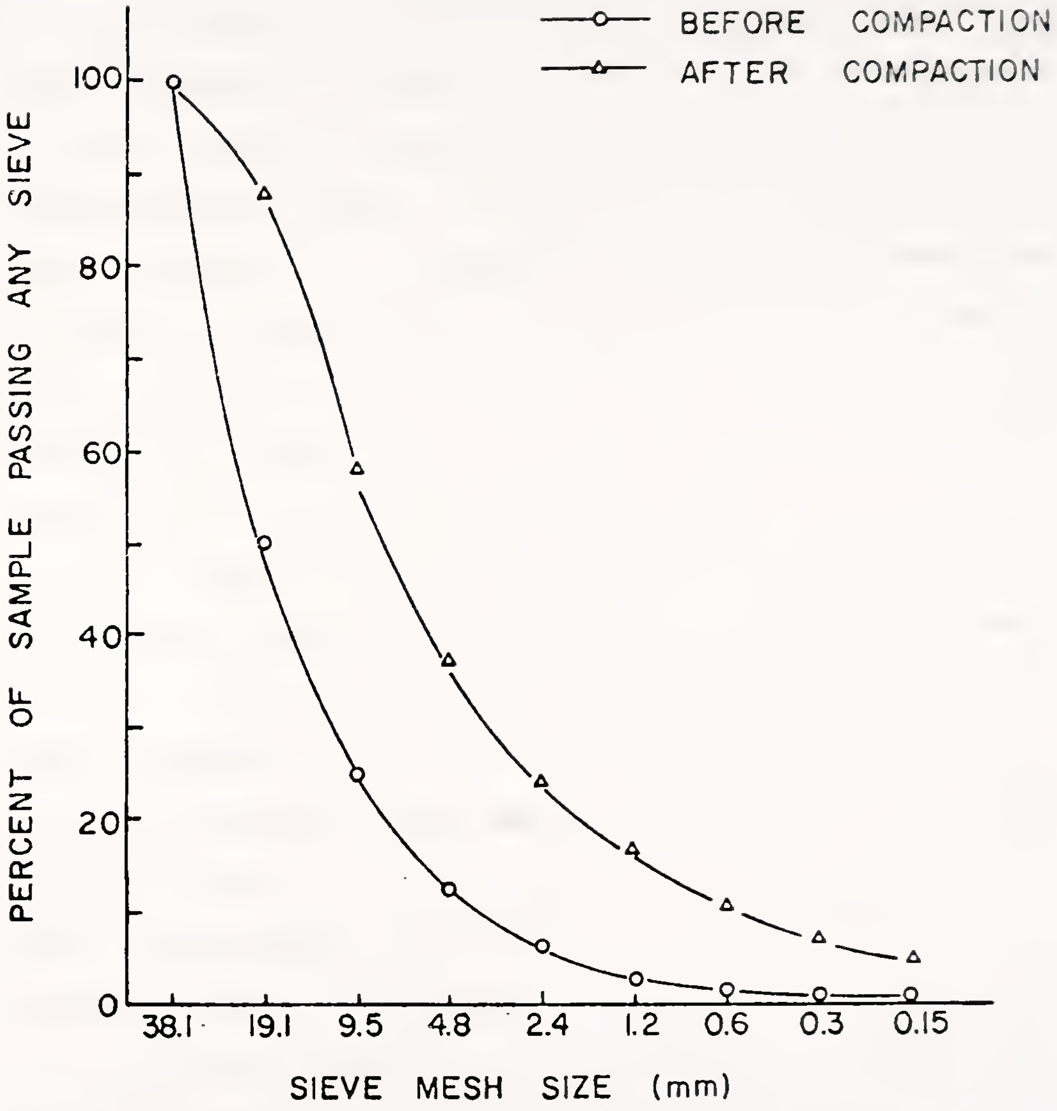


FIGURE 1 INITIAL AND FINAL GRADATIONS OF OSGOOD SHALE SUBJECTED TO IMPACT COMPACTION

coefficient and "coefficient of uniformity" developed by Allen Hazen have gained widespread use in soil mechanics. Bailey (1976) presented an excellent review of these and other gradation coefficients.

The initial and final gradations of a compacted sample could be expressed in terms of the gradiation coefficients. A comparison or ratio of the initial and final coefficients could then represent the change in gradation. However, since each gradation coefficient is a ratio of aggregate sizes, it is possible to have different gradations with identical coefficient values. Also, the coefficients represent only two or three points on the grain size curve. The possibility of misrepresentation and lack of complete information restrict the use of gradation coefficients as measures of degradation.

The limitations of the gradation coefficients indicate the need for numerical gradation values which reflect more complete information. Gradation indices provide methods of representation which satisfy this need. By including the percentage of a sample from each of several size groups, a gradation index can accurately represent an entire grain size curve. Since the actual aggregate size and weight percentage of each size group influences the index value, the ambiguous nature of gradation coefficients is avoided.

The Index of Crushing (IC) is a gradation index based on the summation of the weighted fractions of several size groups. Aughenbaugh et al. (1963) described the use of the IC as a measure of aggregate degradation during compaction. The percentage of the sample by weight within a size range is multiplied by a factor equal to the mean equivalent mesh size of that range. The summation of the values from each

size group represents one gradation. The actual IC value is computed as the difference between the numerical representation of the initial and final gradations and expressed as a percentage of the value from the initial gradation. Table 1 shows an example of the method of calculating the IC used in this study.

Upon inspection, the calculation of the gradation values used for the IC shows a great similarity to the calculation of the commonly used mean or average value in statistics. As explained by Harr (1977) the mean value of grouped data (K groups) is taken as

$$\text{mean} = \frac{\sum_{i=1}^{K} m_i f_i}{\sum f_i}$$

where m_i is the midpoint and f_i is the frequency of group i. With this explanation, the gradation values in the IC represent the mean or average aggregate size of the initial and final gradations. The IC is then a measure of change in the mean aggregate size.

As discussed by Bailey (1976) the IC enables the comparison of samples with dissimilar initial gradations. The IC is based on real measures of aggregate size and weight percentages. According to Bailey

> "When degradation is expressed as the percent change in the gradation index, thus relating both initial and final conditions, the 'real' base of the weighting factors allows direct comparison of samples without the need for scaling or oversize corrections. This enables degradation comparisons between small scale laboratory tests and actual field compaction."

The successful use of the IC by Bailey, the ability to use the IC for samples with different initial gradations, and the concept

Table 1. Method of Calculating the Index of Crushing (after Aughenbaugh et al., 1963)

SIZE GROUP DEFINITIVE SIEVE NUMBERS	DEFINITIVE SIEVE MESH (mm)	MEAN EQUIVALENT MESH SIZE (mm)	PERCENT OF SAMPLE INCLUDED	WEIGHTED FRACTION OF SAMPLE
1 1/2 TO 3/4	38.1 TO 19.1	28.60	A_1	28.60 x A_1
3/4 TO 3/8	19.1 TO 9.52	14.31	A_2	14.31 x A_2
3/8 TO #4	9.52 TO 4.76	7.14	A_3	7.14 x A_3
#4 TO #8	4.76 TO 2.38	3.57	A_4	3.57 x A_4
#8 TO #16	2.38 TO 1.19	1.78	A_5	1.78 x A_5
#16 TO #30	1.19 TO 0.59	0.89	A_6	0.89 x A_6
#30 TO #50	0.59 TO 0.297	0.444	A_7	0.444 x A_7
#50 TO #100	0.297 TO 0.149	0.223	A_8	0.223 x A_8
#100 TO pan	0.149 TO 0.000	0.074 (ASSUMED)	A_9	0.074 x A_9

$$B = \Sigma A$$

B_0 = Sum of Weighted Fractions Before Compaction

B_1 = Sum of Weighted Fractions After Compaction

$$\text{Degradation} = IC = \frac{B_0 - B_1}{B_0} \times 100\%$$

of mean aggregate size led to the use of the IC as the primary measure of degradation for this study.

3.3 Use of the Aggregate Size Distribution

The gradation of a material may also be represented as a differential frequency distribution or histogram of aggregate sizes. This differential frequency distribution displays the percent weight of the sample for each size group. Figure 2 shows an example of the histogram and the approximation to the distribution.

The differential frequency distribution has limitations similar to those of the cumulative distribution curve. Yet, two major features make the former a useful analytical tool. First, when the initial and final gradations are plotted together as aggregate size distributions, the changes in percent weight within one size group can be seen immediately. The comparisons within groups point out aggregate sizes which have been affected most by the compaction process and the specific increases in finer aggregates.

The second feature is the ability to observe the variations of several samples compacted using the same set of variables. By comparing the results of several samples within individual size groups, the variation in the results may be localized. This information can then be used in assessing the significance of the variability in the compaction process.

3.4 Selection and Description of Test Shales

The shale classification system developed by Deo (1972) lists four categories. Based on their durability, shales are classified as

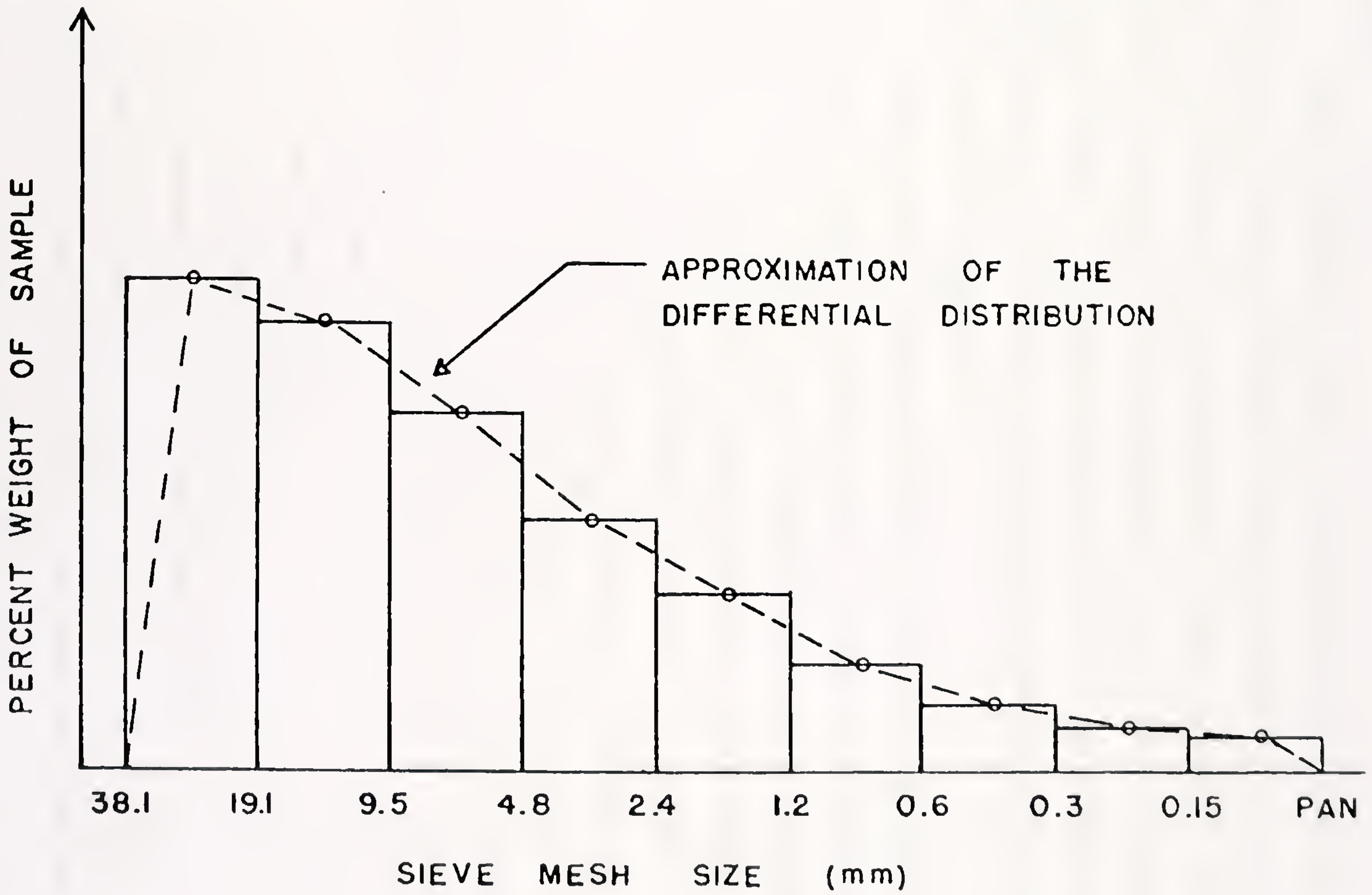


FIGURE 2 FREQUENCY HISTOGRAM OF AGGREGATE SIZE DISTRIBUTION

"rock like", "intermediate 1 or 2", or "soil like" shales. Through an extensive testing program by the Indiana State Highway Commission (ISHC), nearly all Indiana shales have been placed in the soil like category.

Shales which are difficult to break down during compaction but are nondurable present particular engineering problems. For this reason, the ISHC decided to concentrate on shales classified as soil like which were harder and more durable than the weakest shales in the soil like category. The more durable of the soil like shales were characterized by a wet sample 500 revolution slake durability index number greater than 40 but less than 75.

Fifteen shales previously sampled by the ISHC had slake durability numbers within the desired range. Nine of the fifteen shales were rejected due to lack of use in roadway embankments or sampling difficulties. Samples of the remaining six shales were readily available and access to the field formations presented no problems.

A mineralogy study using X-ray diffraction analysis was performed on the clay fractions of each of the six shales. Sample preparation followed the procedure suggested by Kinter and Diamond (1956). Four samples for each shale were prepared and treated as follows:

1. Air dried
2. Oven dried at 550°C
3. Treated with Glycerol
4. Treated with KCl and oven dried at 550°C

The "Simplified X-ray Analysis Guide for Clay Minerals" (Diamond, 1978), shown in Table B1 for the interplanar spacings (d),

was used to interpret the values obtained from the diffraction patterns. The interplanar spacing (d) were calculated from the positions of the diffraction peaks (2θ) by applying Bragg's Law:

$$d = \frac{n\lambda}{2 \text{ Sin } \theta}$$

where d = interplanar spacings in Angstroms (Å)

λ = wave length of radiation in Angstroms (Å)

$n = 1$ for basal (001) peak position

2θ = diffractometer angle for position of peaks in degrees.

Based on the results of the mineralogy study, the ISHC selected three shales, New Providence, Osgood, and Palestine II, for use in the testing program. A summary of the analysis for each shale is given in Tables B2, B3, and B4. The relative proportions of the clay minerals in each shale were estimated using the peak amplitudes as a rough quantitative guide and are displayed in Figure 3. A more complete description of each shale is given below.

3.4.1 New Providence

New Providence shale shown in Figure 4 lies at the base of the Valmeyeran (Osage) series of the Mississippian system. The shale is gray, has medium hardness, and is flaggy.

Similar in mineralogy to the other formations in the Osage series, the New Providence shale contains quartz and feldspar (Harrison and Murray, 1964). As shown in Figure 3, the mineralogy study indicates the clay portion of the New Providence shale is primarily illite with some kaolinite and chlorite.

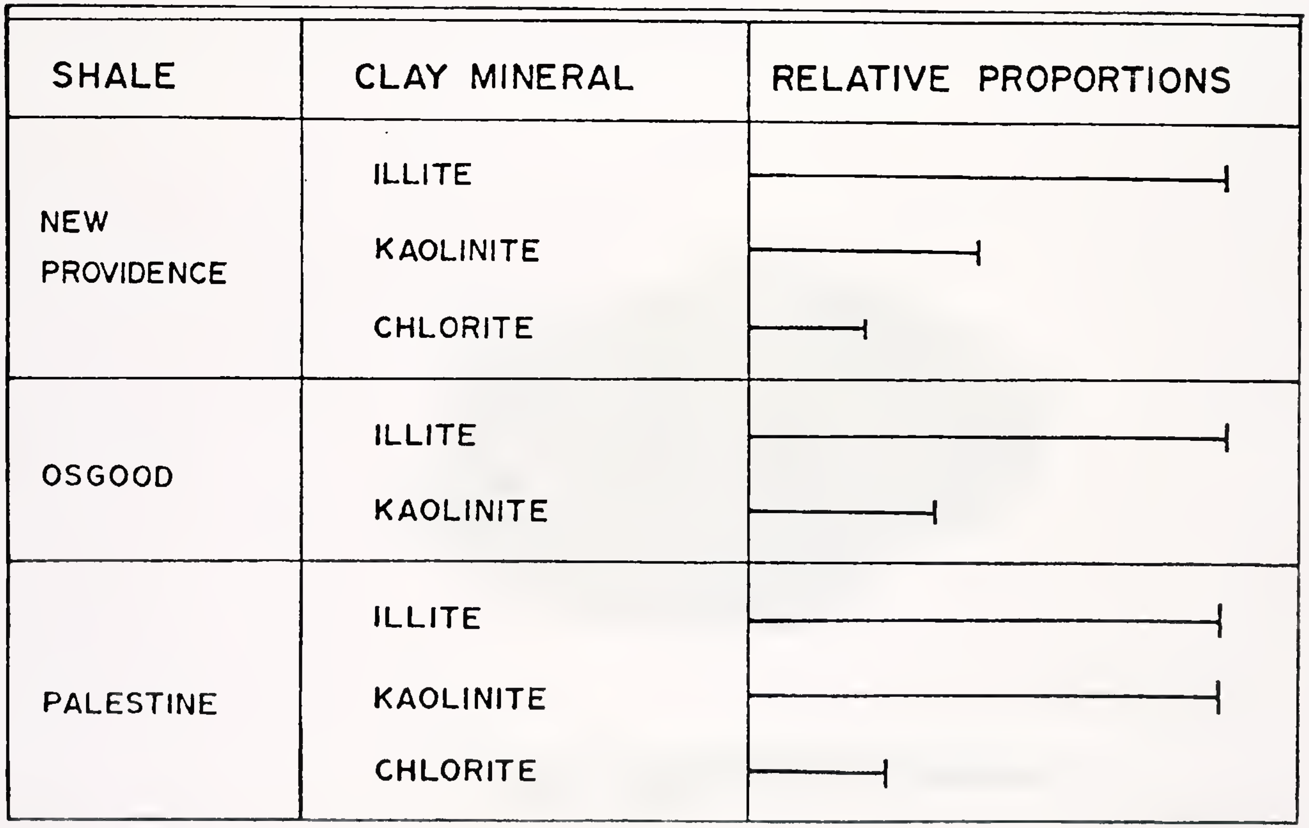


FIGURE 3 ESTIMATED RELATIVE PROPORTIONS OF CLAY MINERALS IN TEST SHALES

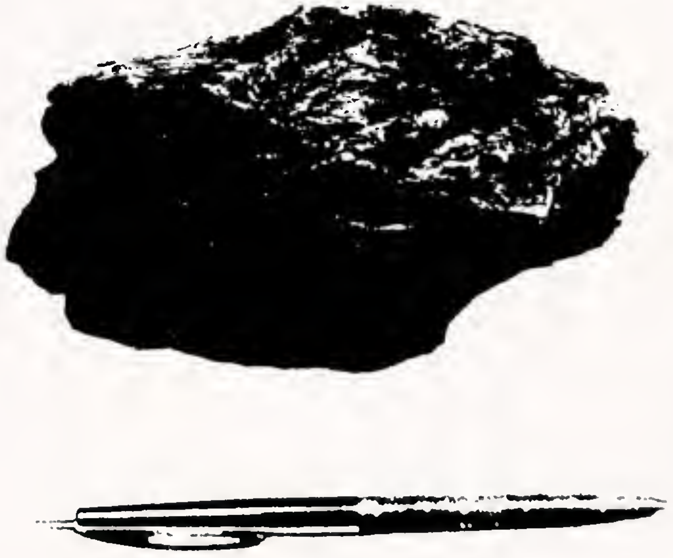


FIGURE 4 SAMPLE OF NEW PROVIDENCE SHALE

The results of tests performed by the ISHC on the New Providence shale are given in Table B5.

The New Providence shale was sampled along the ramp leading from westbound I-64 to northbound I-265 near New Albany, Indiana. The shale was excavated with a backhoe, loaded onto a truck, and covered with a tarpaulin. The following day, the shale was stored in a steel bin at Purdue. A more complete description of the storage container is given in Appendix C.

3.4.2 Osgood

The Osgood shale shown in Figure 5 is a member of the Salamonie dolomite and lies at the base of the Niagaran series in the Silurian system. The Osgood shale is blue-gray, hard, and flaggy. As part of the Silurian system, the Osgood shale represents the oldest material used in the testing program.

The Silurian system is best described as a succession of limestones and dolomites (Harrison and Murray, 1964). As a shale of the Silurian system, the dominant minerals in the Osgood shale are quartz, dolomite, and calcite (Harrison and Murray, 1964). The mineralogy study of the clay portion shows primarily illite with some kaolinite. No chlorite is found in the Osgood shale. The results of tests performed by the ISHC on the Osgood shale are given in Table B6.

The Osgood shale was sampled near the intersection of Indiana SR 107 and US 421 in Madison, Indiana. The material was excavated with a backhoe, loaded onto a truck, and covered with a tarpaulin. The shale arrived at Purdue the following day and was stored in a steel

FIGURE 5 SAMPLE OF OSGOOD SHALE

bin. A more complete description of the storage container is given in Appendix C.

3.4.3 Palestine II

The Palestine shale is part of the Palestine sandstone formation in the Chester series of the Mississippian system. The rocks of the Chester series consist of shales, sandstones, and limestones in relatively thin strata (Harrison and Murray, 1964). The Palestine shale shown in Figure 6 is brown-gray, soft, and flaky and can best be described as a transition between shale and sandstone.

Quartz is the abundant nonclay mineral in the Palestine shale. The mineralogy study on the clay portion shows kaolinite and illite in approximately equal amounts. Some chlorite is also present.

The results of tests performed by the ISHC on the Palestine shale are given in Table B7.

The Palestine shale was sampled from the side of a road cut along the westbound lane of I-64, approximately 56 km (35 mi) west of Corydon, Indiana. Because of the height of the sample pit above the road elevation, the shale was excavated with hand tools and a jackhammer. The samples were placed in cloth bags lined with plastic and sealed. The bags were delivered to Purdue two days later. The shale was then transferred to steel cans with plastic liners. A more complete description of the storage method is given in Appendix C.

3.5 Selection of Compaction Methods

The development of a basic laboratory compaction test by R. R. Proctor in 1933 greatly improved the knowledge of soil compaction.

FIGURE 6 SAMPLE OF PALESTINE SHALE

Other tests were later developed which either modified the Proctor test or presented compaction methods which simulated the action of field compaction equipment. The majority of present compaction tests may be classified as dynamic or impact, static, or kneading. Each of these techniques was considered in the selection of compaction methods for the degradation testing program.

Impact compaction tests basically consist of dropping a hammer a specified number of times on material contained in a mold. Slight modifications of the test used by Proctor have led to the development of several "standard" test procedures (ASTM D-698 and D-1557; AASHTO T-99 and T-180). The impact compaction tests have become a fundamental part of soil testing and are backed by a vast amount of laboratory and field experience.

The Proctor type compaction tests use a hammer with a diameter much smaller than the mold. Another type of impact test developed in England uses a hammer which fits closely inside the mold. This "full-face" impact test (British Standard 812:1960) is used by the British Standards Institution as a standard test for measuring the resistance of aggregates to crushing under impact loads.

Aughenbaugh et al. (1963) and Bailey (1976) used Proctor type compaction methods to induce degradation in sample materials. Using the Modified AASHTO method and a 15.24 cm (6 in) mold, Aughenbaugh et al. measured the degradation of base course aggregates as a function of blows per layer. The results of the laboratory tests were compared with the observations from field compaction tests. Despite some anomalous values, the Modified AASHTO test produced a fair distinction among the general breakdown characteristics of the different aggregates.

However, the percent fines produced by the impact tests for the different aggregates did not directly match the field results. Because of this discrepancy, Aughenbaugh et al. (1963) rejected impact compaction tests as a method of evaluating the degradation of base course aggregates.

Bailey (1976) performed a limited number of tests using Proctor type compaction methods to evaluate the effect of moisture on shale degradation. Several problems were encountered including the inability to dry sieve the moist samples. Because of these problems, Bailey discontinued the use of Proctor type compaction methods.

The degradation tests by Bailey (1976) included the use of full face impact compaction. The results showed that full face compaction produced fairly uniform and repeatable levels of compacted unit weight but that degradation values varied widely. Also, from visual inspection of the compacted samples, the upper portions of the samples had experienced more degradation than the lower portions. Bailey attributed this behavior to the tendency for shale pieces in the upper portions of the sample to "lock up" during the first blows of the compaction hammer. The poor repeatability of degradation values led to Bailey's (1976) rejection of full face compaction as a suitable index test.

As part of a program using coal mine refuse in highway embankment construction, Butler (1976) used Proctor type compaction tests to observe the effect of degradation on the compaction characteristics of coarse refuse material. Repetitive testing of samples showed that increased degradation improved the moisture-density relationships of the material. Butler also reported reasonably good agreement between

laboratory predictions and field results for both degradation and moisture-density relationships. Butler (1976) concluded that use of the Proctor compaction method and the resulting test data could provide improved construction-compaction guidelines.

Although the conclusions by Aughenbaugh et al. (1963), Bailey (1976), and Butler (1976), regarding the use of Proctor type tests in degradation testing were mixed, the simplicity and availability of Proctor tests favored this form of impact testing. Therefore, the Proctor test was selected as a suitable compaction method in the degradation testing program.

Static compaction methods are characterized by the slow application of a full coverage stress to the sample. The majority of experience in the use of static tests for aggregates come from England and South Africa.

The aggregate crushing test of the British Standards Institute (British Standard 812:1960) uses a 400 kN (45 ton) load to crush aggregates confined in a steel mold. The crushing value is measured by the amount of fines produced by the loading and is expressed as a percentage of the total sample weight.

The large load required by the aggregate crushing test makes the test generally insensitive to the strength of weaker materials. To overcome this problem Shergold and Hosking (1959) developed the 10% Fines, Aggregate Crushing Test (10-FACT). The 10-FACT (British Standard 812:1960) is expressed as the load required to produce 10% fines in the sample. Since the applied load is variable, weaker materials may be tested successfully.

Loubser (1967) reported that the 10-FACT was a reliable method of evaluating indurated shales used as road aggregates in South Africa. Case histories were presented which verified the capabilities of the 10-FACT in predicting the performance of shale aggregates. Additional work by Loubser (1967) showed a good correlation between the 10-FACT and the density and water absorption characteristics of shales.

The results of static tests performed by Bailey (1976) were very consistent in terms of load, Index of Crushing, and density. Because of the good correlations found in the static testing, Bailey recommended the static test as a standard degradation index test.

In view of the success associated with static testing, this method of compaction was included in the testing program.

Kneading compaction methods attempt to model the actions of field compaction equipment. Unlike impact or static compaction, the compactive effort of the kneading compactor occurs in cycles of gradually increasing and decreasing load and is applied through a compaction foot. The compactive effort may be controlled by the foot pressure and the number of applications per sample. Detailed descriptions of the compaction process and kneading apparatus are given by Aughenbaugh et al. (1963), Bailey (1976), and Gaudette (1960).

Aughenbaugh et al. (1963) used kneading compaction as a method of evaluating the degradation of base course aggregates. The results of the kneading compaction tests were compared with observations from field compaction and other laboratory methods. The kneading compactor failed to differentiate correctly among the various aggregates with respect to their general breakdown characteristics. Also, variations

in compactive effort from changes in the foot pressure or number of applications produced erratic and unpredictable amounts of degradation. Based on the results obtained, Aughenbaugh et al. (1963) rejected the kneading compactor as a degradation index test.

Bailey (1976) performed kneading compaction tests to study the degradation of shale. In the testing program, the number of applications and compaction foot pressure were varied independently. The general trends of the results showed reasonable relationships between degradation, density, and compactive effort. However, Bailey (1976) reported that, "In general it was found that repetitions of kneading compaction tests with all conditions held constant did not yield the same results."

The anomalous results reported by Aughenbaugh et al. (1963) and Bailey (1976) indicated that kneading compaction was not a sufficient compaction method as an index test for shales. For this reason, kneading compaction was excluded from the testing program.

3.6 Testing Procedure

3.6.1 General

The impact and static methods of compaction demonstrated features suitable for a standard compaction-degradation test. A testing program was developed to examine the repeatability of the compaction process involved in each method. Samples of the New Providence, Osgood, and Palestine shales were prepared in an identical manner to reduce the test variation. The assumption was made that by using similar samples, any variation in the test results was largely due to the shale

variability and testing process. The shale variability was a factor present in all tests. The view was taken that the variation due to the material would be at a constant level throughout the testing program. Thus, any major differences among the variations found within the individual effort levels or compactive techniques were attributed to the compaction process.

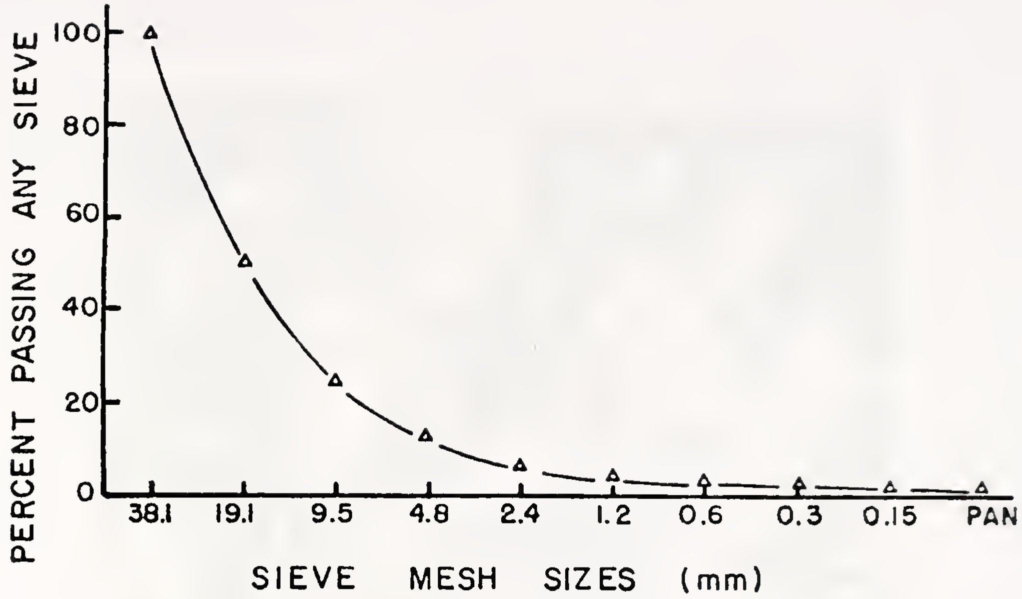
The use of four levels of compactive effort in both the impact and static tests permitted one to demonstrate the effects of compactive effort on degradation and compacted density. Different effort levels also allowed for the evaluation of specific compactive efforts in terms of the limits established by the guidelines of the testing program.

3.6.2 Sample Preparation

The excavation methods used to obtain the shale produced a number of large pieces. The large pieces were broken down with a carpenter's hammer to prepare samples suitable for testing. The broken shale was then dry sieved through a nest of sieves with mesh sizes of 38.1, 19.1, 9.52, 4.76, 2.38, 1.19, 0.59, 0.30, and 0.15 mm and a pan (U.S. Standard sieves 1 1/2 in, 3/4 in, 3/8 in, No. 4, 8, 16, 30, 50, and 100). The material from each different size group was stored in a plastic bag.

A 5.0 kg (11.0 lb) sample was prepared immediately before testing by blending the different sizes to fit the distribution of gradation A shown in Figures 7, 8, and A1. The cumulative distribution of gradation A conformed to the equation:

$$P = \left({}^{d}/_{D}\right)^{1}$$



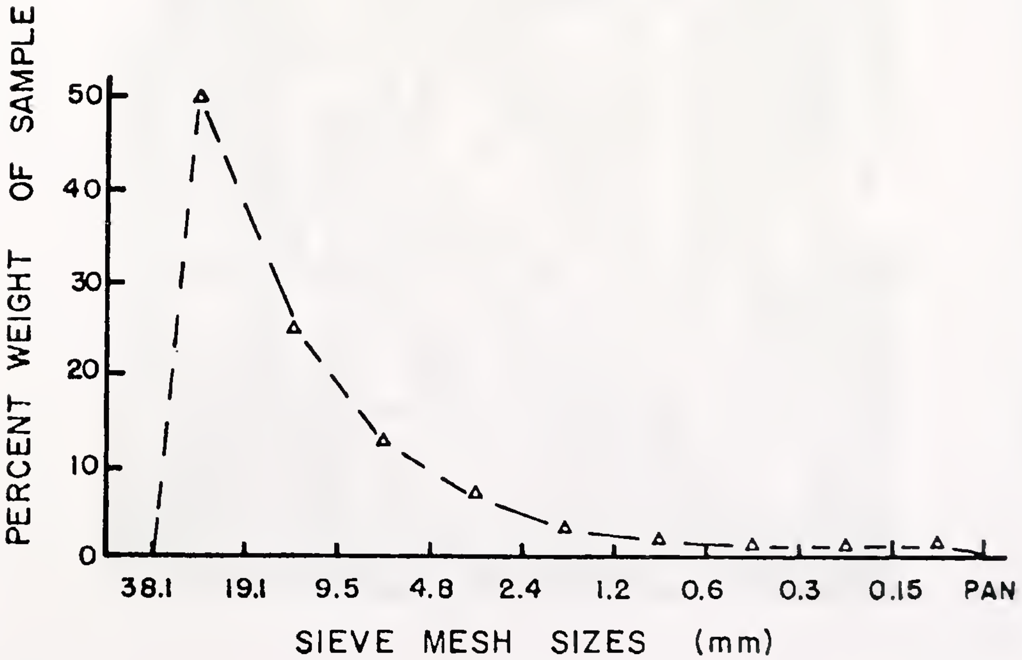


FIGURE 7 CUMULATIVE AND AGGREGATE SIZE DISTRIBUTIONS OF INITIAL GRADATION A

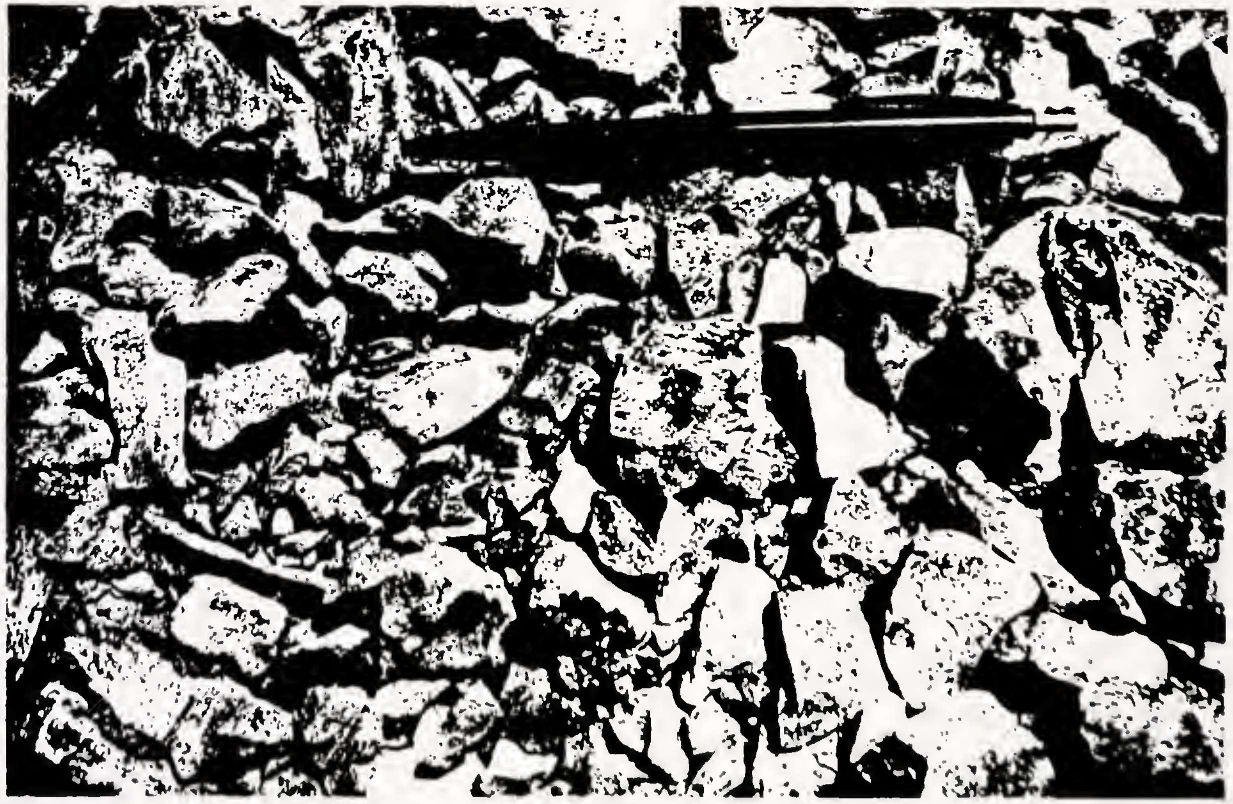


FIGURE 8 SHALE SAMPLE BLENDED TO
FIT GRADATION A

where P is the percent passing any sieve, d is the sieve mesh size, and D is the top aggregate size.

The gradation of the samples used by Bailey (1976) was similar to the above relationship and involved a top aggregate size of 19.1 mm (0.75 in). A top size of 38.1 mm (1.5 in) was used for the samples in this testing program in an attempt to provide a more realistic representation of field conditions. The gradation equation provided a size distribution which emphasized the larger pieces but still included the finer sizes. The combination of the selected gradation curve and the larger sizes was expected to more accurately reflect the behavior of shale degradation during compaction.

The shales were tested at their natural moisture levels to avoid the effects of either wetting or drying. Although the values of moisture content varied among the three shales, the moisture contents for samples from any particular shale were relatively uniform.

3.6.3 Impact Tests

The impact tests were similar to the Proctor-type compaction procedure in that a small faced hammer was dropped on the sample material a specified number of times. The equipment and compactive efforts were modified to satisfy the special needs of the testing program.

A 15.24 cm (6 in) diameter steel CBR mold was used to accommodate the top aggregate size of the sample gradation. Bailey (1976) reported problems due to the additional degradation induced while removing the more tightly compacted shale samples of higher effort levels. To

alleviate this problem and aid sample removal, all samples were compacted in the split CBR mold shown in Figure 9.

The different levels of compactive effort were controlled by the weight and drop of the hammer, the number of layers, and the number of blows per layer. Table 2 provides a summary of the combinations of variables used to obtain each effort level.

A 5.1 cm (2 in) thick, 15.0 cm (5.9 in) diameter disk was placed in the mold prior to compaction. The disk effectively reduced the volume of the mold so that less material was required for each test. The shale was then placed in the mold and compacted at the desired effort level.

After compaction the mold collar was taken off. Next, any excess material above the mold rim was gently removed. For the calculation of wet density, the shale remaining in the mold was removed and weighed on a scale accurate to 0.005 kg (0.01 lb).

The compacted sample was separated by hand and recombined with the excess material that had been previously removed. All of the shale was dry sieved through the nest of sieves used in the sample preparation. A more complete description of the sieving process is given in Appendix C. After sieving, the material from each of the size groups was weighed on a scale accurate to 0.1 gm. A portion of the sample material was oven dried at 105°C for one week to determine the sample moisture content.

The determination of the Index of Crushing (IC) was accomplished by the method shown in Table 1 once the initial and final sample gradations were known. The practice of recombining the shale before the

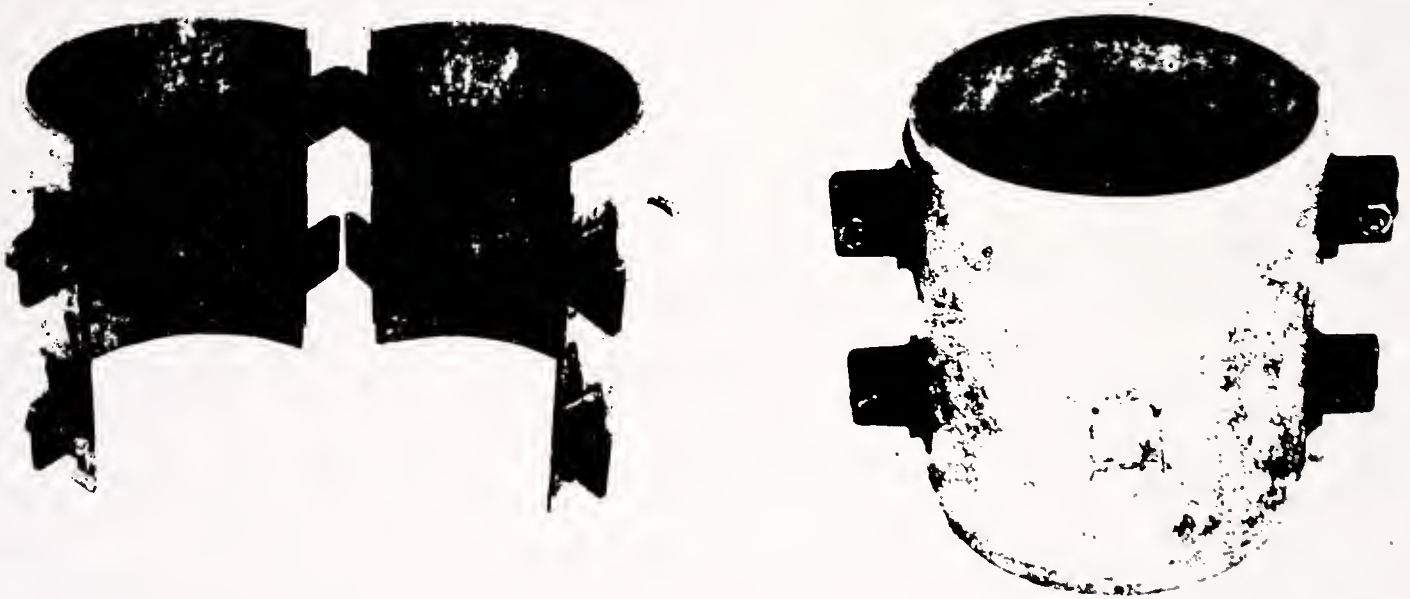


FIGURE 9 SPLIT MOLD USED IN COMPACTION TESTS

Table 2. Summary of Compactive Effort Variables

Effort Level	Total Compactive Effort $kN\text{-}m/m^3$ ($ft\text{-}lb/ft^3$)	Weight of Hammer kg (lb)	Drop Hammer cm (in)	Number of Layers	Blows per Layers
1	527 (11,100)	2.49 (5.5)	30.5 (12.0)	3	55
2	790 (16,500)	4.54 (10.0)	45.7 (18.0)	3	30
3	1451 (30,300)	4.54 (10.0)	45.7 (18.0)	3	55
4	2414 (50,400)	4.54 (10.0)	45.7 (18.0)	5	55

final sieving insured the analysis of the entire sample. Including all the sample in the IC calculation reduced possible errors due to the elimination of portions of the sample gradation.

3.6.4 Static Tests

The static tests were characterized by the slow application of a load distributed over the entire face of the sample. As in the impact tests, modifications in equipment and procedures were developed to satisfy the special requirements of the static testing. The split CBR mold and disk from the impact tests were also used in the static testing. A loading ram was constructed to transfer the compressive effort to the sample. The ram shown in Figure 10 consisted of a 6.5 cm (4.0 in) diameter hollow steel cylinder with 15.0 cm (5.9 in) diameter circular plates welded on each end. The total height of the ram was 14.0 cm (5.5 in).

The static compaction test had an advantage over the impact test in the determination of compactive energy. The total compactive work done on a sample could be calculated by measuring the load during the compaction process and the residual deformation of the sample. A dial gage was attached to the loading ram to evaluate the deformation of each statically compacted sample. The compactive effort in the static tests varied only in the highest load applied to the sample. The four levels used are given below:

1. 500 kPa (72.5 psi or 2000 lb)
2. 1000 kPa (145.0 psi or 4015 lb)
3. 2000 kPa (290.0 psi or 8030 lb)
4. 3000 kPa (435.0 psi or 12045 lb)

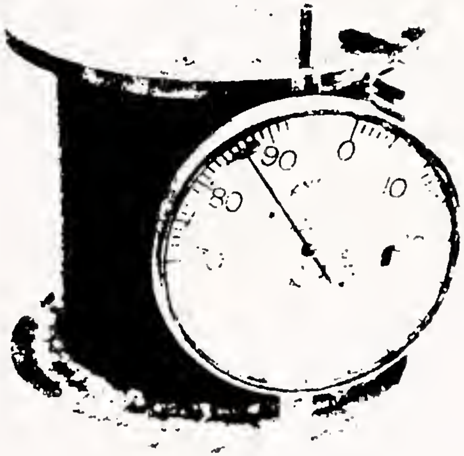


FIGURE 10 LOADING RAM USED IN
STATIC COMPACTION TESTS

Each sample was compacted in three layers. A loose layer of material was placed in the mold, then the loading ram was positioned and seated. The entire assembly was then placed in a compression testing machine for loading (see Figure 11). Although the lowest setting on the machine was used for all the tests, the loading rate varied between 0.15 cm/min (0.06 in/min) and 0.25 cm/min (0.10 in/min). This variation was attributed to the hydraulic system of the machine and was not considered a significant factor in the testing program.

The load was increased to the desired level and then released immediately. Following compaction, the sample was trimmed, weighed, and dry sieved using the procedure described for the impact tests.

The load and deformation were monitored for each layer. Values from the compression machine load gage were recorded at regular time intervals during the compaction process. As the load approached the higher values, the slope of the load-time curve increased sharply (see Figure 12).

The sample deformation, measured by the loading ram dial gage, displayed the linear deformation-time relationship shown in Figure 12 during loading. The deformation showed a sharp decrease upon release of the load which indicated the elastic rebound of the sample. Only the residual deformation, measured as the difference in the dial gage reading before and after loading, was used in calculating the work input. Since the elastic portion of the total deformation was assumed to have a linear relation with load, the relationship of the residual deformation and the load was also linear.

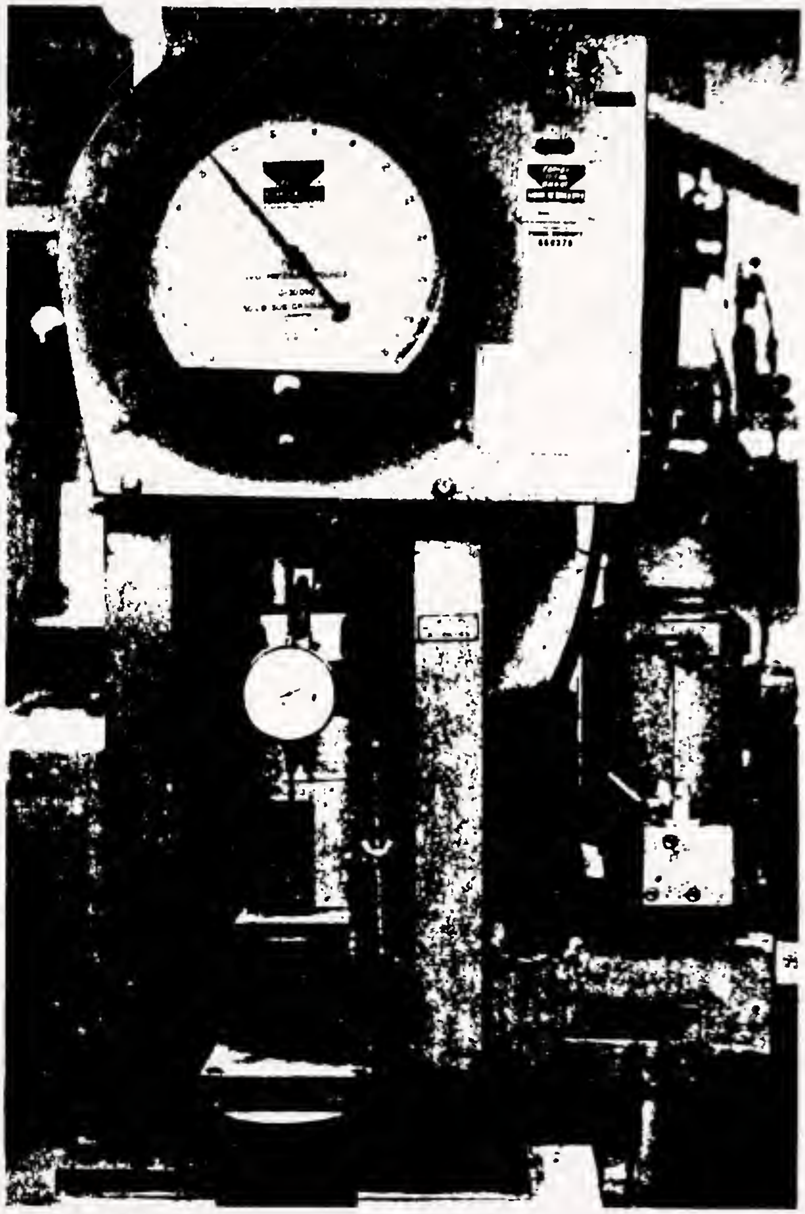


FIGURE 11 COMPRESSION MACHINE FOR STATIC COMPACTION TESTS

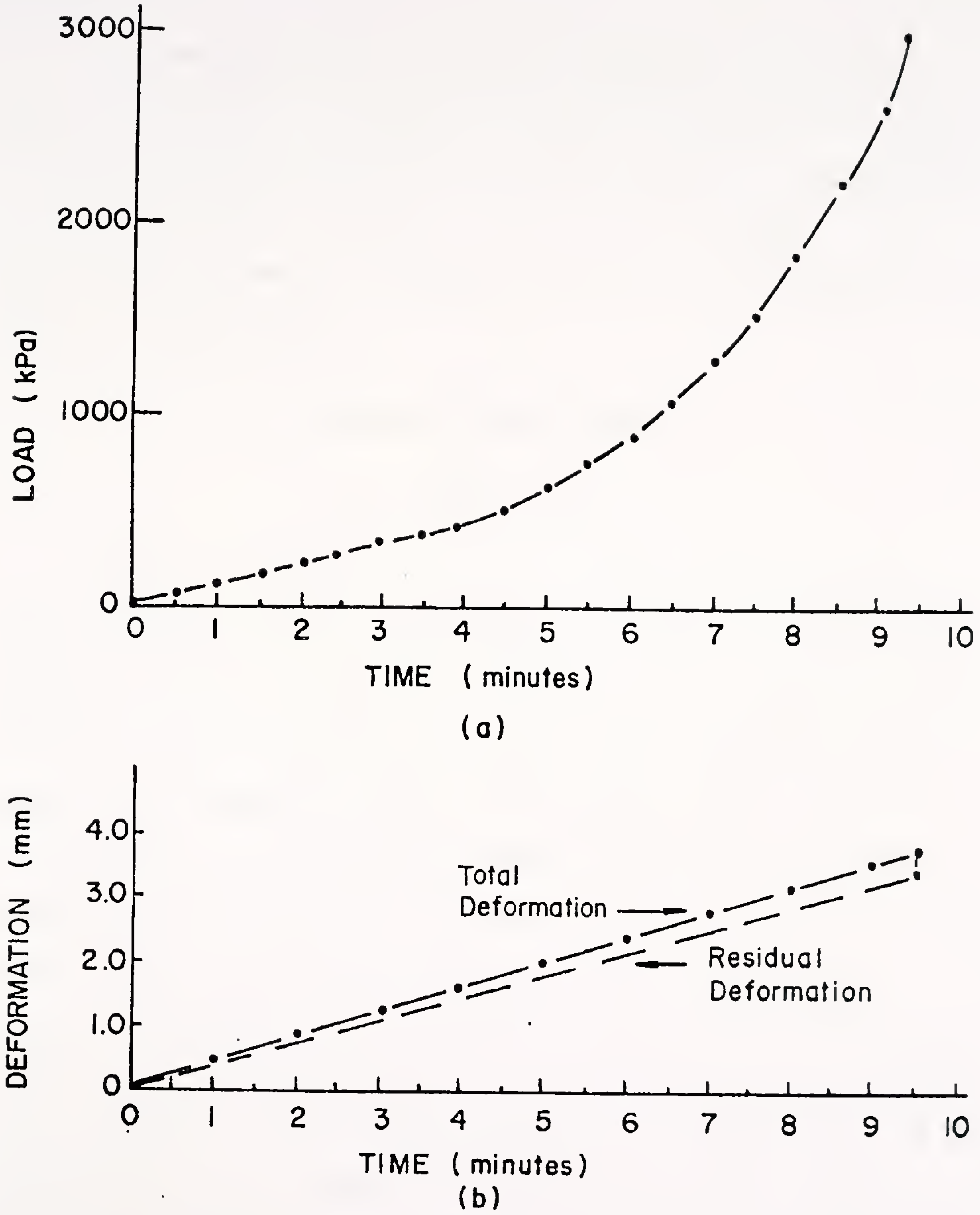


FIGURE 12 RELATIONSHIPS OF LOAD AND DEFORMATION WITH TIME

The work input could be calculated by using the trapezoidal method to estimate the area beneath the load-residual deformation curve. The total compactive work input for a sample was taken as the summation of the work that had been applied to each layer in the sample. This quantity was normalized by multiplying by the ratio of the mold volume to the final sample volume.

3.7 Results of Testing Program

3.7.1 Impact Tests

Impact tests using four levels of compactive effort were performed on each of the three shales. As seen in Figure 13, degradation generally increased with increasing compactive effort. The relationship between degradation and compactive effort generally agreed with the results of shale degradation tests performed by Bailey (1976). Bailey also reported a limiting maximum value of degradation with increasing compactive effort. No limiting degradation values were observed for the shales within the compactive efforts used in these impact tests.

The Palestine shale at the third compactive effort of 1450 kN-m/m^3 (30,300 ft-lb/ft^3) deviated from the trend of increasing degradation. The smaller Index of Crushing value was attributed to problems with separation of the compacted sample before dry sieving. The coherence of the Palestine shale compacted at the third effort level created a sample that could not be separated without inducing an unknown amount of degradation. Keeping the effort of separation to a minimum increased the probability of representing a lump containing

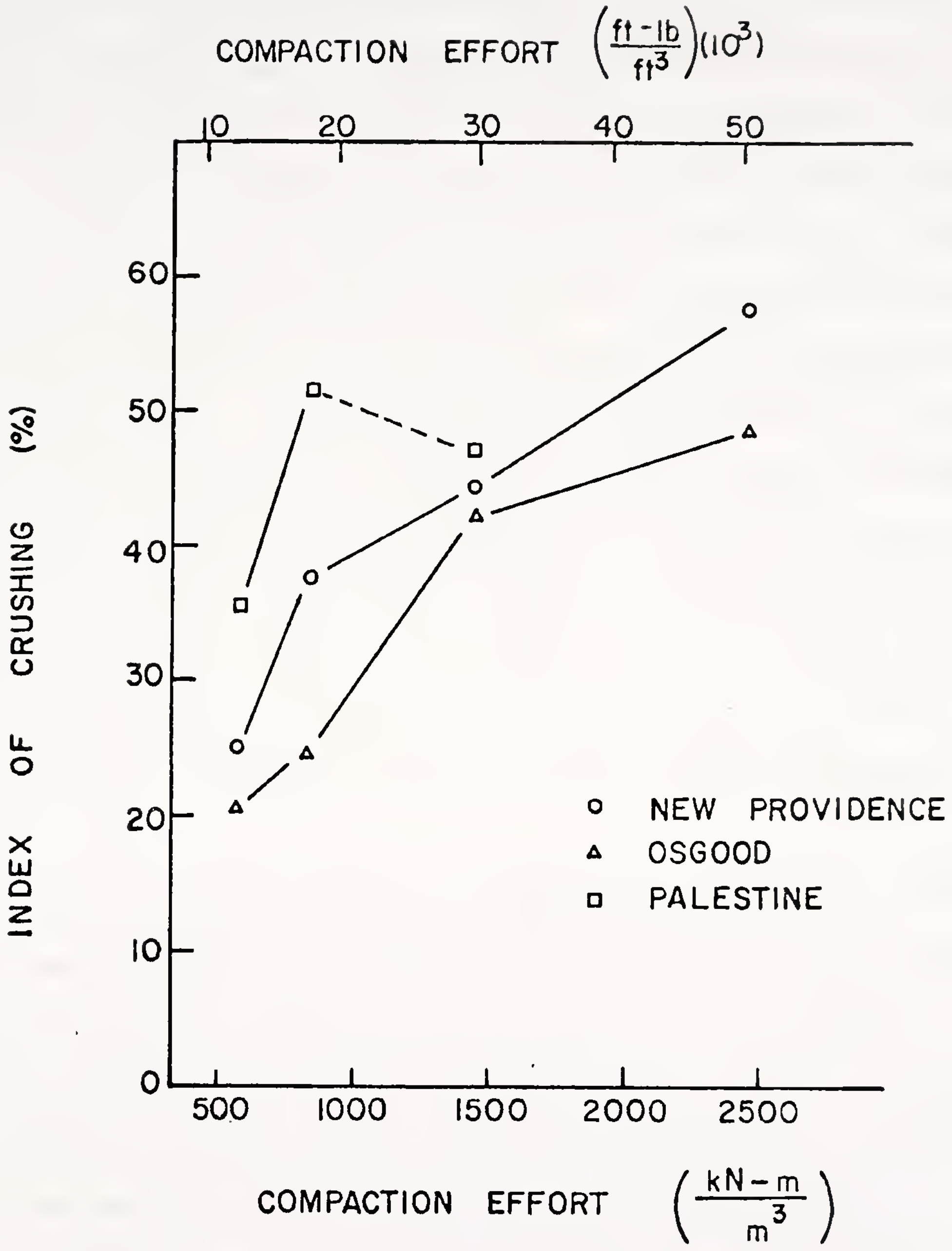


FIGURE 13 EFFECT OF COMPACTION EFFORT ON INDEX OF CRUSHING ; IMPACT COMPACTION

several pieces as a single aggregate size. The problems encountered with the Palestine samples at the third effort level prevented the degradation analysis of the Palestine shale at any higher effort level.

Figures A2 through A12 display the aggregate size distributions of the impact compaction samples and provide an important key in understanding the degradation pattern of the shales. Aggregates in the 38 mm (1.5 in) to 19 mm (0.75 in) size group experienced the greatest percent weight change. Fragments produced by the breakdown of the large pieces were distributed over the entire size range, thereby increasing the amounts of smaller sizes. Aggregates in the medium size range also degraded. However, to some extent, fragments from larger aggregates replaced the broken, medium size aggregates. The overall degradation process produced a final differential frequency distribution which was flatter than the initial distribution and had a smaller mean aggregate size.

The dry density shown in Figure 14 also increased with increasing compactive effort. The compaction tests reported by Bailey (1976) indicated limiting maximum values of dry density as the compactive effort increased. Although no actual limits were reached in this testing program, the density curves for each of the shales show a tendency to become asymptotic at the higher levels of compactive effort.

A direct comparison of the dry density values for the three shales at any given compactive effort is misleading due to the differences in specific gravity among the shales. The use of the percent solids as defined below:

$$\text{PERCENT SOLIDS} = \frac{\text{VOLUME OF SOLIDS}}{\text{TOTAL VOLUME}}$$

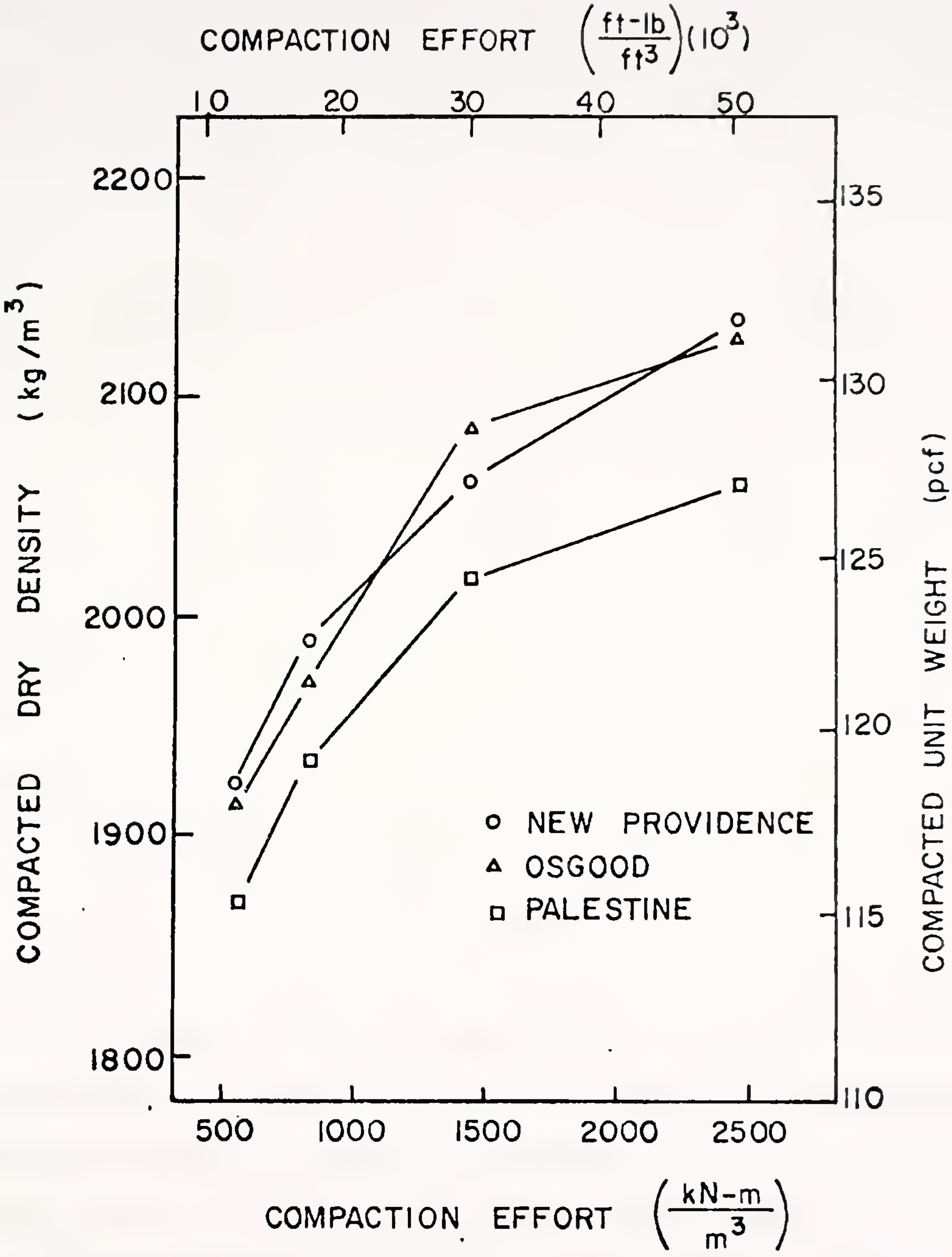


FIGURE 14 EFFECT OF COMPACTION EFFORT ON COMPACTED DRY DENSITY ; IMPACT COMPACTION

provides a measure which corrects for the difference in specific gravity. Table 3 lists the values of percent solids for the shales at each compactive effort level.

Table 3. Average Values of Percent Solids for Impact Compaction Samples

	EFFORT LEVEL			
Shale	1 527 $\frac{\text{kN-m}}{\text{m}^3}$	2 790 $\frac{\text{kN-m}}{\text{m}^3}$	3 1451 $\frac{\text{kN-m}}{\text{m}^3}$	4 2414 $\frac{\text{kN-m}}{\text{m}^3}$
NEW PROVIDENCE	68%	71%	73%	76%
OSGOOD	67%	69%	73%	75%
PALESTINE	68%	70%	73%	75%

Note: $1 \frac{\text{kN-m}}{\text{m}^3} = 20.9 \frac{\text{ft-lb}}{\text{ft}^3}$

The values in the table indicate that for a given effort level and a constant mold volume, the volume of solids is nearly identical regardless of the shale type. An accompanying conclusion is that the volume of voids is also identical at a given effort level.

The degradation of the material during compaction resulted in different final gradations for each shale. As explained previously, degradation is characterized by a reduction in the amount of large aggregates and an increase in the amount of smaller sizes.

The voids (pores) within a compacted shale sample are directly influenced by the size and arrangement of solid aggregates. Therefore, the final gradation of solids in a sample greatly affects the sizes of pores within the sample. In cases where the total volume of voids is constant, the measure of degradation also serves as an indicator of pore size distribution. The results from investigations by Garcia-Bengochea (1978) and Witsman (1979) concerning the influence of pores on the behavior of compacted materials indicate that the pore size distribution has an important role in the permeability, compressibility, and stability of a compacted shale embankment.

The importance of the pore size distribution suggests the need for a conjugate solid size distribution. The percent weight term of the aggregate size distribution is not directly compatible with the void volume function used in the pore size distribution. The percent weight term is easily transformed into a volumetric function by dividing each weight by the shale's specific gravity and the unit weight of water. However, as shown below, the end result is a volumetric value which is identical to the percent weight value.

WITH: W_1 = WEIGHT OF SOLIDS (SIZE GROUP 1)

W_T = TOTAL WEIGHT OF SOLIDS

G = SHALE SPECIFIC GRAVITY

γ_w = UNIT WEIGHT OF WATER

THEN:

$$\text{PERCENT WEIGHT (SIZE GROUP 1)} = \frac{W_1}{W_T}$$

$$\text{VOLUME OF SOLIDS (SIZE GROUP 1)} = V_{S1} = \frac{W_1}{G\gamma_w}$$

$$\text{VOLUME OF SOLIDS (TOTAL)} = V_{ST} = \frac{W_T}{G\gamma_w}$$

$$\text{PERCENT OF SOLID VOLUME (Size Group 1)} = \frac{V_{S1}}{V_{ST}} = \frac{W_1/G\gamma_w}{W_T/G\gamma_w}$$

$$= \frac{W_1}{W_T}$$

This development shows that the distribution for the percent of solid volume is identical to the aggregate size distribution.

The volumetric function for the solid size distribution may be desired in terms of the total sample volume. This is accomplished by multiplying each percent of solid volume value by the percent solids value for the compacted sample.

The variability of the compaction process was indicated by examining the results of tests repeated four times at each effort level. The coefficient of variation (the ratio of the standard deviation to the mean and expressed as a percent) provided an appropriate measure of variability. Table 4 lists the mean Index of Crushing value and the coefficient of variation for each effort level.

The lowest compactive effort (level 1) consistently displayed the greatest variation. The variation decreased as the compactive effort increased to effort levels 2, 3, and 4. The Palestine shale did not follow this trend. The variation at level 3 for the Palestine

Table 4. Index of Crushing and Coefficient of Variation for Impact Compaction Samples

SHALE		EFFORT LEVEL $\frac{kN\text{-}m}{m^3}$	INDEX OF CRUSHING MEAN VALUE (%)	COEFFICIENT OF VARIATION (%)
NEW PROVIDENCE	1)	527	25.7	10.4
	2)	790	37.6	5.6
	3)	1451	43.9	5.0
	4)	2414	57.7	3.9
OSGOOD	1)	527	20.6	20.2
	2)	790	25.1	4.4
	3)	1451	42.1	1.9
	4)	2414	48.6	3.3
PALESTINE	1)	527	36.0	6.7
	2)	790	51.8	1.7
	3)	1451	46.9	10.0

NOTE: $1 \frac{kN\text{-}m}{m^3} = 20.9 \frac{ft - lb}{ft^3}$

shale was larger than the variation at levels 1 and 2 due to the problems encountered with sample separation.

The aggregate size distributions, Figures A2 through A12, help explain the observed variation. The variation in the Index of Crushing value reflects the variability within particular size groups over the entire size range. The distributions for effort level 1 show the greatest variability within the 38.1 mm (1.5 in) to 9.5 mm (0.38 in) size range and significant variation within the 9.5 mm (0.38 in) to 0.3 mm (0.01 in) range. For effort levels 2, 3, and 4, the variation decreases and is generally concentrated in the 38.1 mm (1.5 in) to 4.8 mm (0.19 in) size range.

The variability in the testing process and the shale accounts for the small range and relatively low values of the coefficient of variation observed for the higher effort levels. The high variation of the lowest effort level reflects more than testing or material variability and is mainly due to the relationship between the mechanics of aggregate breakage and the forces produced by the compaction process. The delivery of the compactive effort creates loading conditions which cause compressive, shearing, bending, and torsional stresses. Individual aggregates fail when their ability to withstand the stresses is exceeded. The loads are transferred within the layer through the contact points between aggregates. Thus, the distribution of contact points in the sample directly affects the influence of the compaction process on single aggregates.

The breaking and rearrangement of aggregates during compaction create new contact points which transfer additional stresses to

aggregates which may have previously experienced only limited loading. The loads at the lowest effort level caused some degradation, but did not produce a sufficient increase in contact points to establish a uniform distribution of contact points within the sample. Thus, some aggregates based on their random position within the sample, experienced only minimum stresses and did not fail.

The 38.1 mm (1.5 in) to 4.8 mm (0.19 in) range in the size distributions for the compacted shale sample represents the largest and heaviest single aggregates. The variations in percent weight observed for the larger sizes reflect the influence of a relatively small number of aggregates. Therefore, the variation of the low effort level reflects differences in the number of large aggregates which survived the compaction process.

The more uniform distribution of contact points created by the effort levels 2, 3, and 4 probably insured that every large aggregate would be significantly loaded. Since every aggregate was influenced by the compactive forces, the variation in degradation decreased for samples compacted at these higher levels.

3.7.2 Static Tests

The results of the static compaction tests shown in Figures 15 and 16 repeated the general trends of the impact tests. The degradation values of the Osgood and New Providence shale increased with increasing compactive effort and displayed no limiting value within the given compactive effort range. The Palestine shale degradation also increased with increasing compactive effort, but at a decreasing rate. The asymptotic behavior of the Palestine degradation indicated a

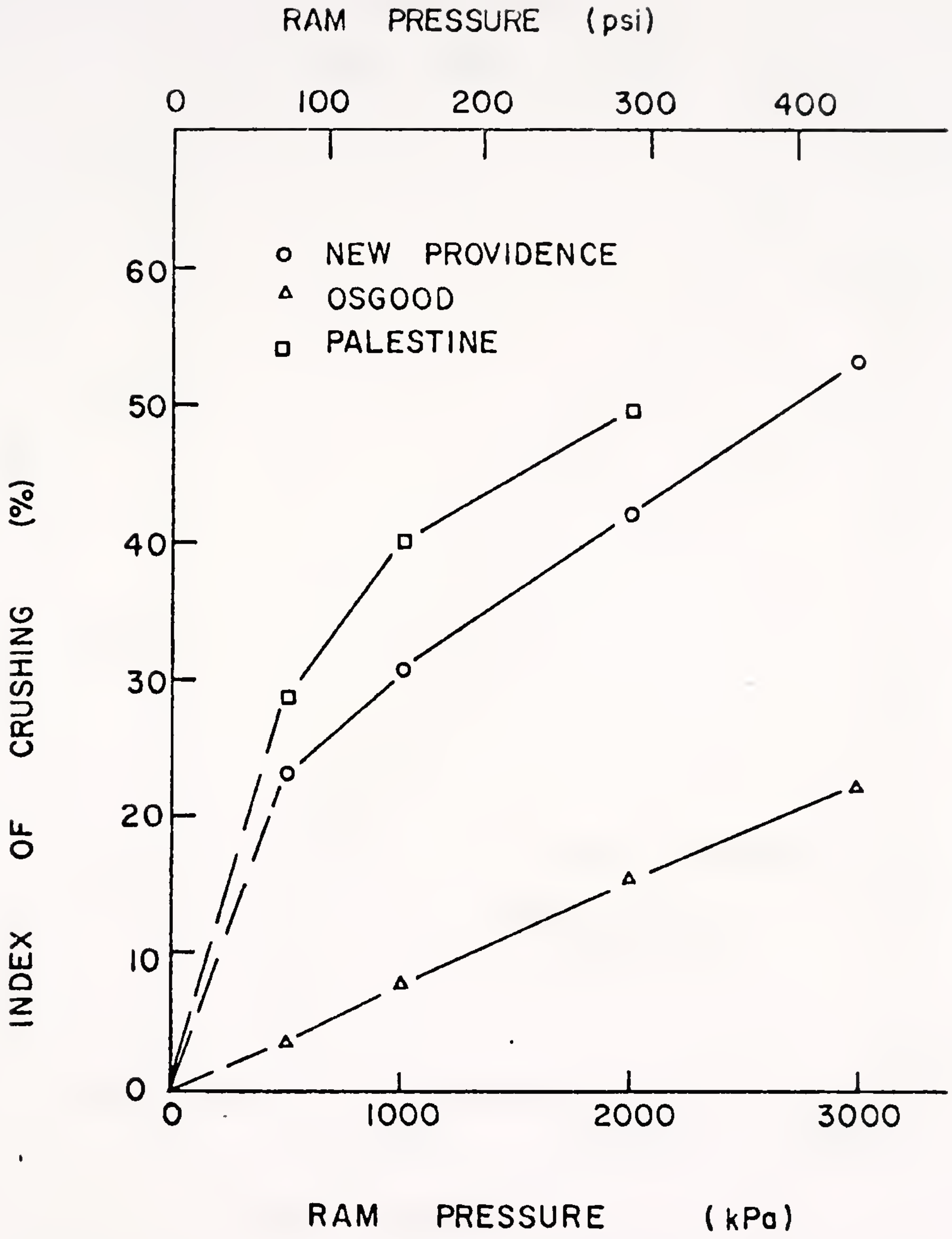


FIGURE 15 EFFECT OF COMPACTION EFFORT ON INDEX OF CRUSHING ; STATIC COMPACTION

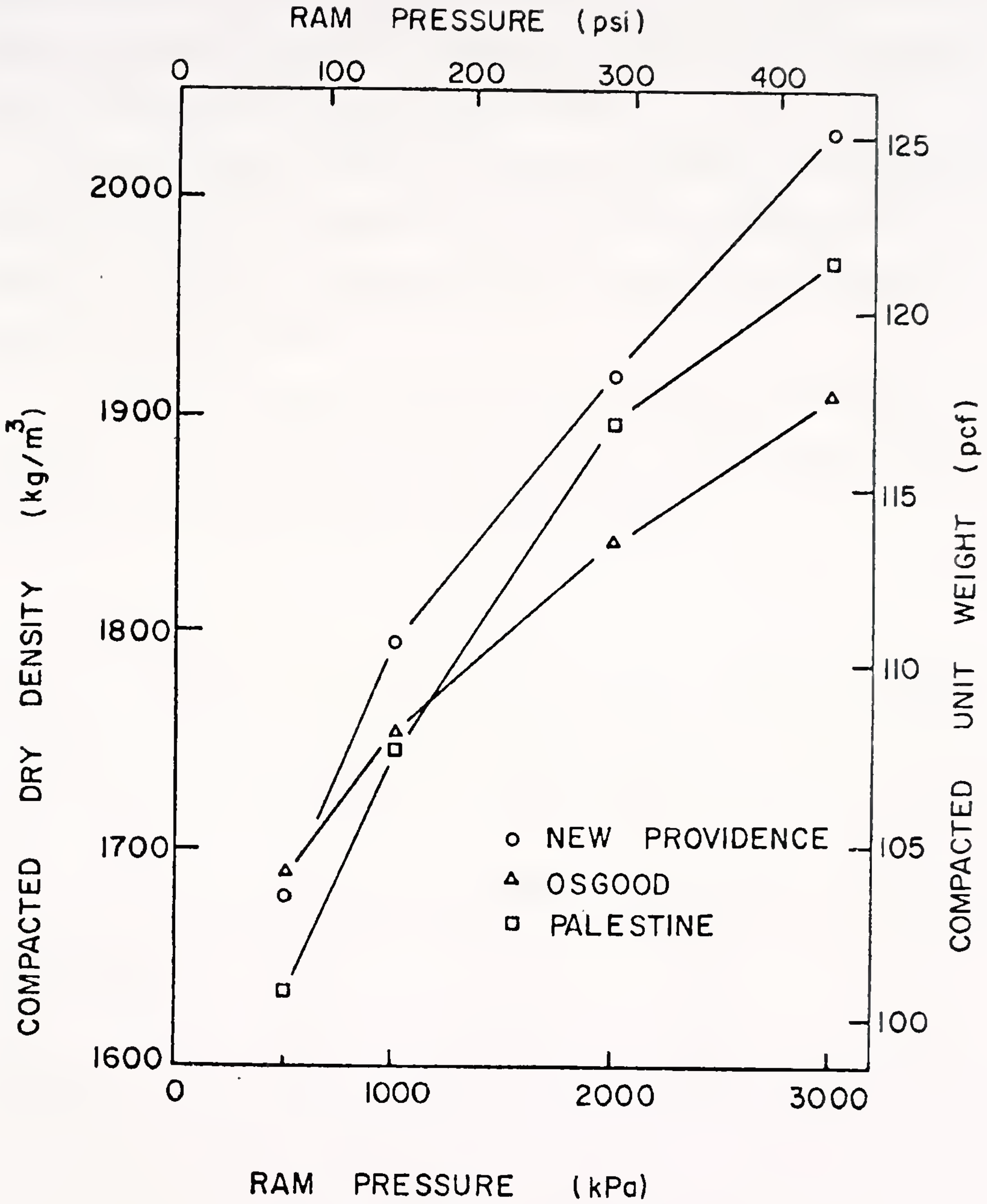


FIGURE 16 EFFECT OF COMPACTION EFFORT ON COMPACTED DRY DENSITY ; STATIC COMPACTION

limiting maximum value as suggested by Bailey (1976). Problems with sample separation prevented the degradation analysis of the Palestine shale at the 3000 kPa (435 psi) effort level.

The dry density of the shale samples increased with increasing compactive effort. The density for all three shales demonstrated the tendency to increase at a decreasing rate, indicating the possibility of maximum density values.

The percent solids values from the static tests (see Table 5) are similar at each effort level, but display a slightly larger range than the percent solids values of the impact tests.

Table 5. Average Value of Percent Solids for Static Compaction Samples

	EFFORT LEVEL			
SHALE	1 500 kPa	2 1000 kPa	3 2000 kPa	4 3000 kPa
NEW PROVIDENCE	60%	64%	68%	72%
OSGOOD	59%	62%	65%	67%
PALESTINE	59%	63%	69%	71%

The application of the compactive effort during the static tests was more closely controlled than for the impact tests due to the ability to regulate the loads applied by the compression machine. Only two tests were performed at each static level because the results were expected to show less variation than the impact test results.

The aggregate size distributions of the statically compacted samples as depicted in Figures A13 through A23, illustrate the variation found between the final gradations of duplicate samples tested at the same effort level. The largest variation for the New Providence and Palestine shales occurred at the lowest effort level. The variation decreased to insignificant amounts as the effort increased. The observed trend of variation in the New Providence and Palestine shales duplicates the variation in the impact tests and can be attributed to the distribution of contact points within the sample.

The static test results of the Osgood shale deviated from the basic trend in variation. The Osgood samples showed only a minimal amount of variation at each effort level. However, the small amount of degradation for the Osgood samples at the low effort levels resulted in final gradations that were very similar to the initial gradations. The final gradations of the samples were nearly identical since the same initial gradation was used in each sample.

The compactive work input was measured by finding the area under the load-residual deformation curve for each statically compacted sample. The hypothesis was adopted that a measurement of the work actually performed on a sample would give a better insight into the relationships between the compactive effort, dry density, and degradation.

The plot of degradation vs. compactive work input (Figure 17) is similar to the plot of degradation vs. ram pressure (Figure 15). The basic shapes of the curves in the two figures are the same although the data points in Figure 17 are spread out more along the abscissa.

The use of the compactive work input as the abscissa for the density plot (Figure 18) allows the placement of data points at intermediate levels which did not exist for the ram pressure. The spread of data points along with abscissa results in a more narrow bandwidth of density values at specific levels of work input.

The density-work relationship becomes nearly linear for the shales when the differences in specific gravity are considered. The use of the percent solids term corrects for the differences in specific gravity and produces the relationship shown in Figure 19. Linear regression analysis performed on the work vs. percent solids data points resulted in the following equation.

$$\begin{matrix}\text{PERCENT}\\ \text{SOLIDS}\end{matrix} = \left[0.59 + 0.00014\ (\text{WORK})\right] \times 100 \qquad R^2 = 0.95$$

A comparison of the density-ram pressure function and the density-work function emphasizes the differences in expressing the compactive effort as a nominal energy or as compactive work.

The loads applied during the compaction process lead to an increase in sample deformation and a corresponding increase in density. Both load and deformation may be related to density as shown in Figure 20. The total sample deformation will eventually reach a maximum as the loads continue to increase. No further densification will occur after this point is reached and a corresponding limiting value for

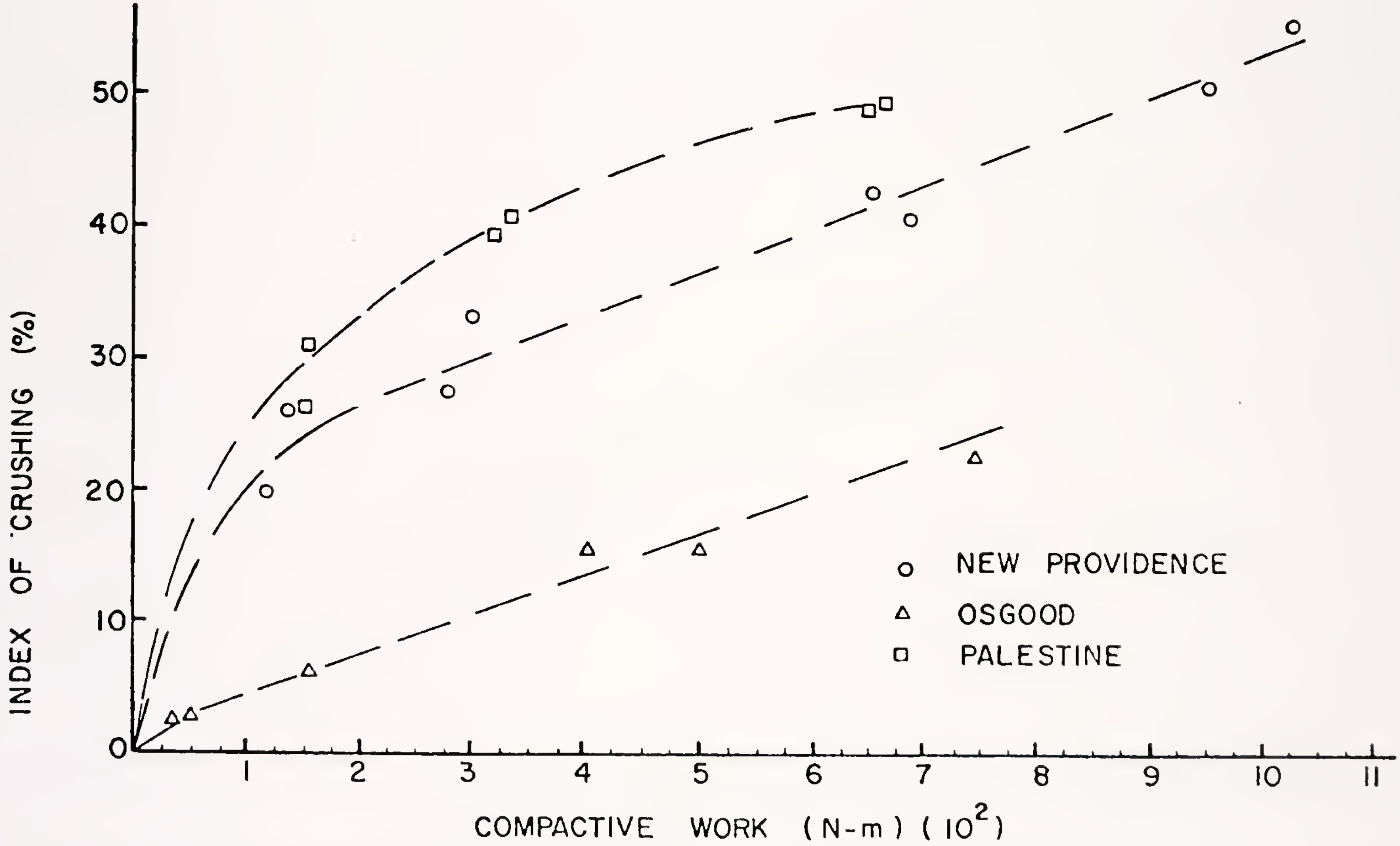


FIGURE 17 COMPARISON OF INDEX OF CRUSHING AND COMPACTIVE WORK

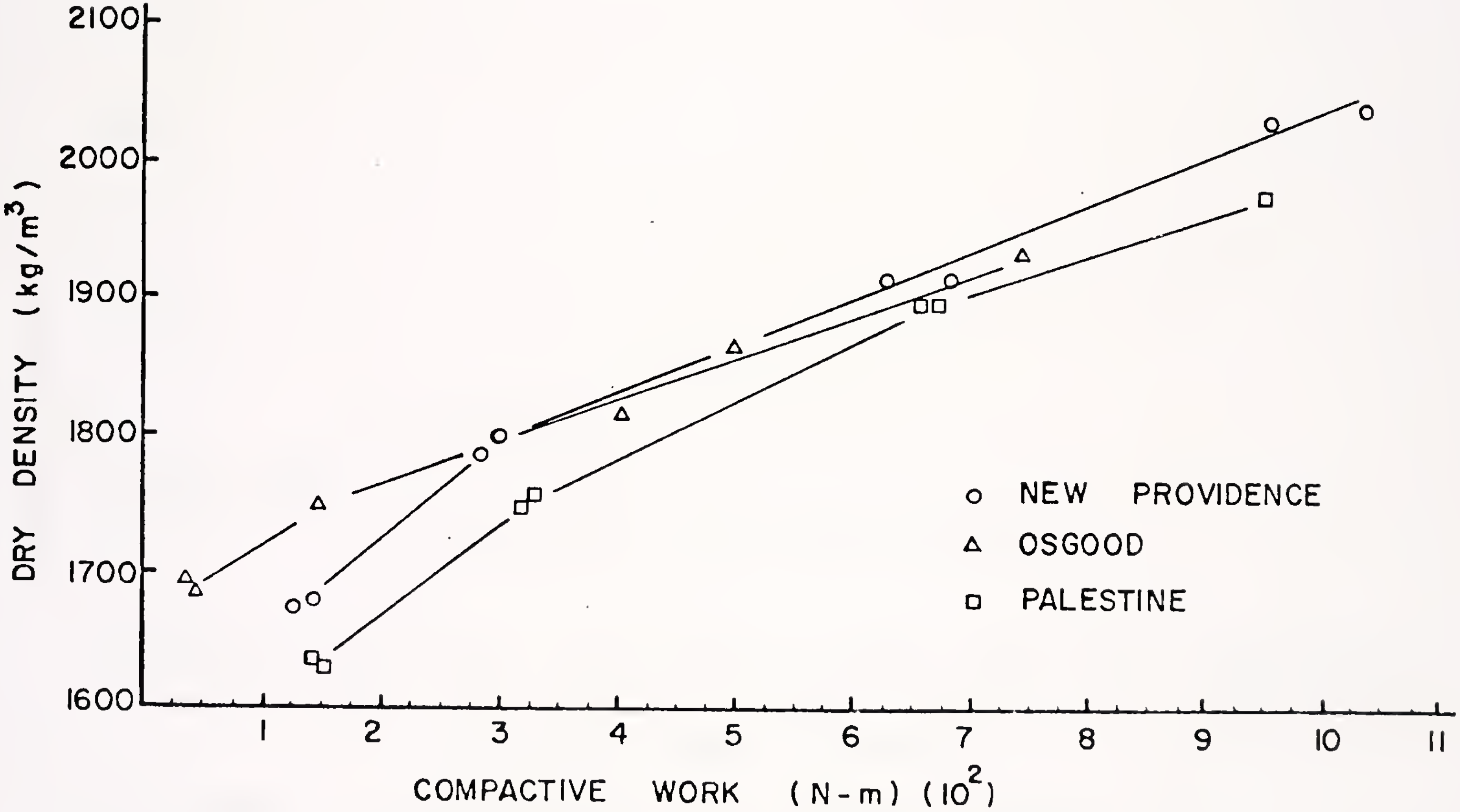


FIGURE 18 COMPARISON OF DRY DENSITY AND COMPACTIVE WORK

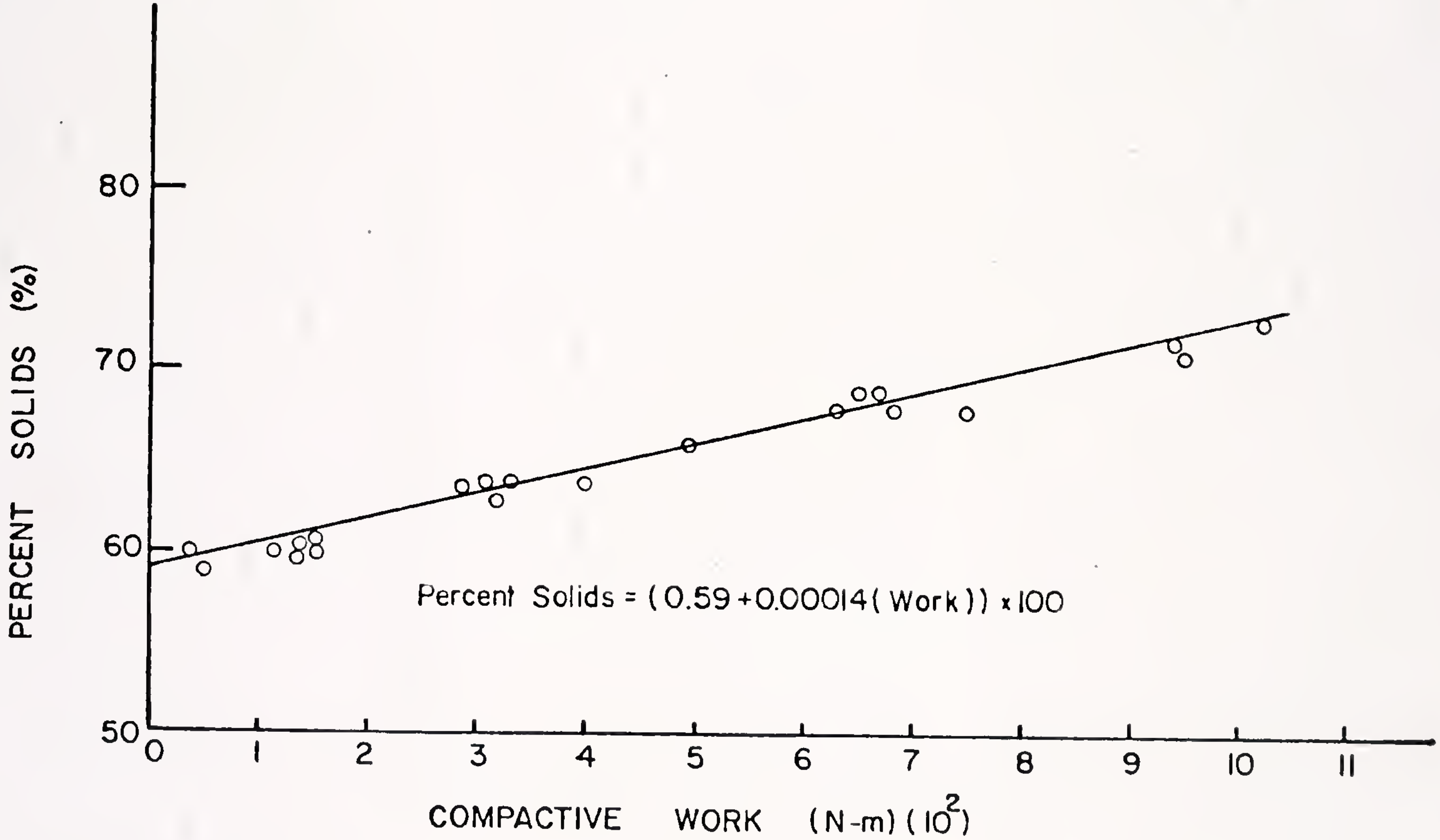


FIGURE 19 RELATIONSHIP OF PERCENT SOLIDS TO COMPACTIVE WORK FOR ALL SHALES

density is approached. Therefore, after maximum deformation (and maximum density) has been reached, compactive energy is wasted.

The nominal energy expression of compactive effort includes only the component of applied load. Increasing the load increases the compactive effort but will have no effect on the density if maximum deformation has been reached. The use of a nominal compaction energy expression will yield a density-effort curve which will approach a maximum density value asymptotically. The density-ram pressure relationship from the static tests represents such a curve in the transition from minimum to maximum densities.

The compactive work expression is a product of load and deformation. Increased loading will not increase the compactive work once the sample deformation has stopped. Therefore, the limiting value of compactive work is reached when the maximum deformation and density are obtained. As seen from Figure 18 the density and compactive work relationship is nearly linear and will continue in a linear manner until the limiting value is reached.

3.8 Conclusions and Selection of a Standard Test

Both the impact and static tests possess the simplicity and availability that is desired for a standard test. The tests also displayed sufficient repeatability at the moderate and high effort levels. The relationships between compactive effort, dry density, and degradation established by each compaction method were similar, although not identical. Based on the above conclusions, either form of testing could serve as a standard test.

The unique features offered by each testing method were evaluated before one test was selected over the other. The main advantage of the static test was the ability to measure the compactive work. The linear relationship between work and density, as well as the concept of limiting work and density values indicate the compactive work expression of effort is more logical than the expressions of nominal compactive energy currently used. Unfortunately, the measurement of compactive work during field compaction is difficult to obtain and is generally neglected in favor of terms describing the compaction equipment and number of passes.

A comparison of the density-degradation relationship shown in Figure 21 reveals higher values of dry density for the impact samples than for static samples at equivalent levels of degradation. The higher density values for the impact tests reflect the amount of aggregate movement that occurred during the compaction process. This increased movement gave the pieces a greater opportunity to establish a more dense packing. Aggregate movement in the static compaction was more restricted. Because of this behavior, large aggregates would fracture during the static loading, but the resulting fragments would essentially remain in place. The increased degradation would have little effect on the dry density without the rearrangement of the broken fragments.

Although neither the impact nor the static tests directly model field compaction, the aggregate movement of the impact test will more closely approach the behavior of shale during field compaction. The impact test also has the advantage of being a well-known and accepted procedure in geotechnical laboratories and is backed by a vast amount of experience.

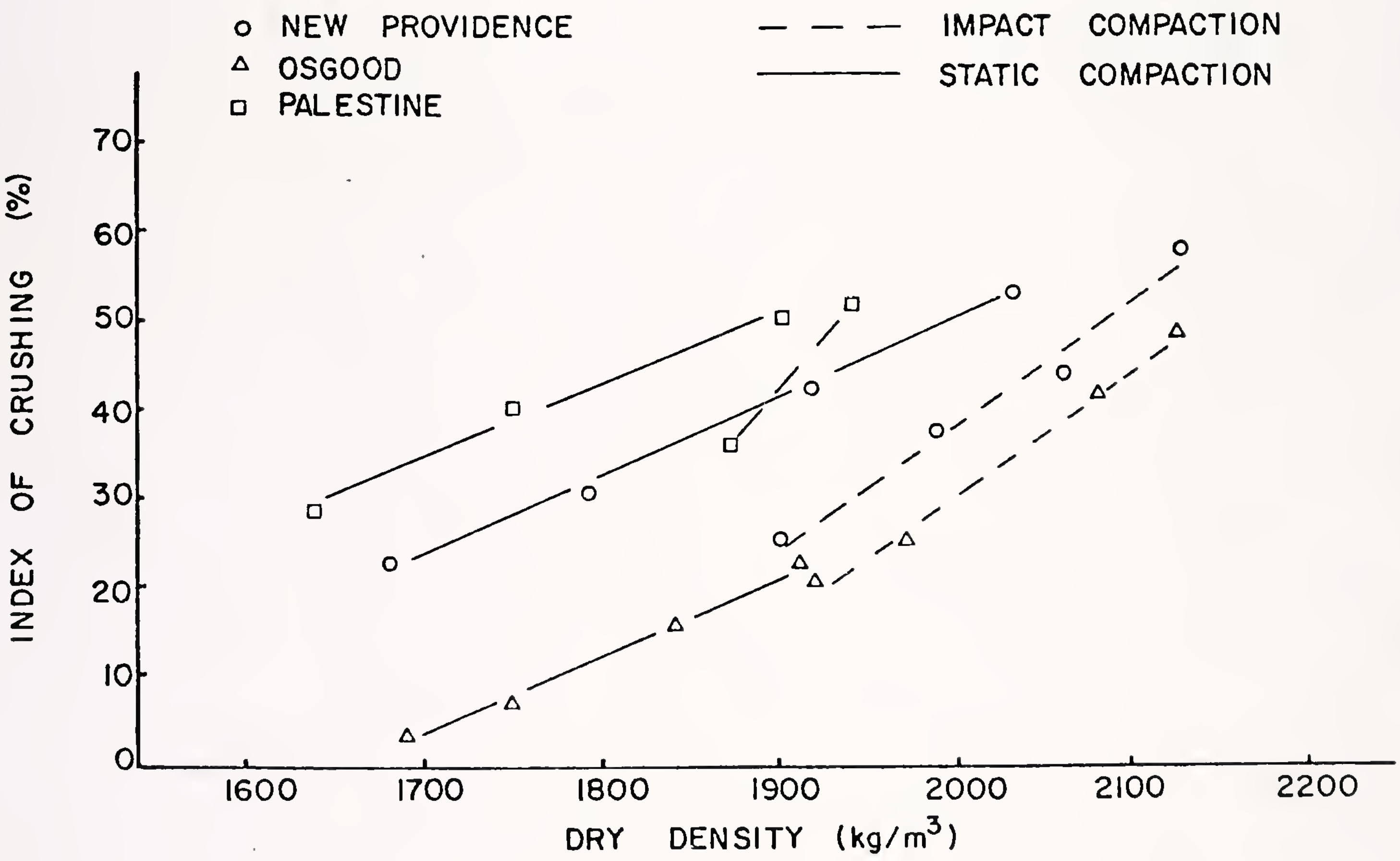


FIGURE 21 COMPARISON OF DENSITY-DEGRADATION RELATIONSHIP FOR IMPACT AND STATIC COMPACTION

The advantages of the compactive work expression favor static compaction as a research tool. However, the results of this testing program indicated that the performance and background of the impact test favor this form of compactive effort. These reasons, along with the previous discussions, led to the selection of the impact test as the preferred compaction method for a standard test.

The four levels of compactive effort used in the impact tests provided an opportunity to observe the effect of compaction energy on degradation and dry density. However, only one effort level was desired to evaluate the effect of other compaction variables or to classify the degradability of shales. The lowest effort level consistently displayed the greatest variation in the test results. The higher compactive efforts, levels 2, 3, and 4, produced results with sufficient repeatability. However, levels 3 and 4 proved to be too severe for the softest shale. For these reasons, effort level 2 (790 $\frac{\text{kN-m}}{\text{m}^3}$ or 16,500 $\frac{\text{ft-lb}}{\text{ft}^3}$) was selected as the most appropriate effort level for the impact test.

In summary, the impact test method at the 790 $\frac{\text{kN-m}}{\text{m}^3}$ (16,500 $\frac{\text{ft-lb}}{\text{ft}^3}$) effort level was selected as the standard compaction-degradation test for shales. The test could then be used to evaluate the effect of compaction variables on shales or as a classification test for the degradability of shales.

4. FACTORS AFFECTING DEGRADATION AND DENSITY

4.1 The Effects of Initial Gradation and Maximum Size on Degradation and Dry Density

4.1.1 Review

Factors other than the compactive effort will affect the degradation and density characteristics of a compacted shale. The more important variables for the case of a shale embankment are the initial gradation, maximum size, and shale moisture content. The standard compaction-degradation test developed earlier was used to evaluate each of these variables independently.

Previous experience with the effect of grading and maximum size on degradation was developed primarily in the field of mineral aggregates used for highway construction. Aughenbaugh et al. (1963) stated that base course aggregates under loading would approach a maximum density gradation through degradation. Less degradation would occur as the initial sample gradation became closer to that for a maximum density gradation. Lees and Kennedy (1975) also reported that the degradation of low strength aggregates decreased as the initial gradation became more densely graded.

The results of kneading compaction tests by Bailey (1976) on shale demonstrated the effect of maximum size. Bailey used two uniform

gradations with top sizes of 19.1 mm (0.75 in) and 4.8 mm (0.19 in). The gradation with the largest top size displayed more degradation at all levels of compactive effort than the other gradation. Ramana Sastry (1977) performed a laboratory study on the degradation of four gradings; two skip-graded and two continuously graded. He reported that the skip gradings degraded more than the continuous gradings, and that a reduction in the maximum size or the amount of the largest size fraction decreased degradation.

The initial gradation and maximum size of a material can also be expected to influence the dry density of a compacted sample. The results of research by Butler (1976) on coal mine refuse demonstrated that the density increased as the material became more well-graded through compaction induced degradation. The kneading compaction tests by Bailey (1976) displayed the effect of maximum size on dry density. The compacted unit weights of two uniform gradations with top sizes of 19.1 mm (0.75 in) and 4.8 mm (0.19 in) were compared over a range of compactive efforts. At the lower effort levels, the unit weights were very similar. However, the differences in the unit weights increased as the compactive effort increased.

4.1.2 Procedure for Analyzing the Effects of Initial Gradation

Three different gradings were used to observe the effect of initial gradation on degradation and dry density. Gradation A was used in the development of the standard test. The cumulative distribution curve of gradation A followed the equation:

$$P = (d/D)^{1}$$

where P is the percent passing any sieve size, d is the sieve mesh size, and D is the maximum aggregate size. By selecting a maximum size of 38.1 mm (1.5 in), results from previous tests could be applied to the analysis of initial gradation.

Gradation B also followed an exponential cumulative distribution curve expressed as;

$$P = (d/D)^{0.5}$$

where P, d, and D are described above. Gradation B had a better distribution of sizes than gradation A.

Gradation C represented a very narrow-banded grading and consisted entirely of aggregates in the 38.1 mm (1.5 in) to 19.1 mm (0.75 in) size range. The cumulative distributions of the three gradings are displayed in Figure 22.

One sample of gradations B and C was prepared for each shale type. The shales were compacted at their natural moisture contents in accordance with the procedure developed for the standard test. The samples were trimmed, weighed, and dry sieved using the procedure described in Section 3.6.3 and Appendix C. Moisture content evaluations were made for each sample to determine the dry density.

4.1.3 Results and Discussion of Initial Gradation Effects

The effect of the initial gradation on degradation is illustrated in Figure 23. The gradings are compared by their coefficients of uniformity. The coefficient of uniformity (C_u) is expressed as the ratio of D_{60}/D_{10} where D_{60} and D_{10} are the aggregate size diameters

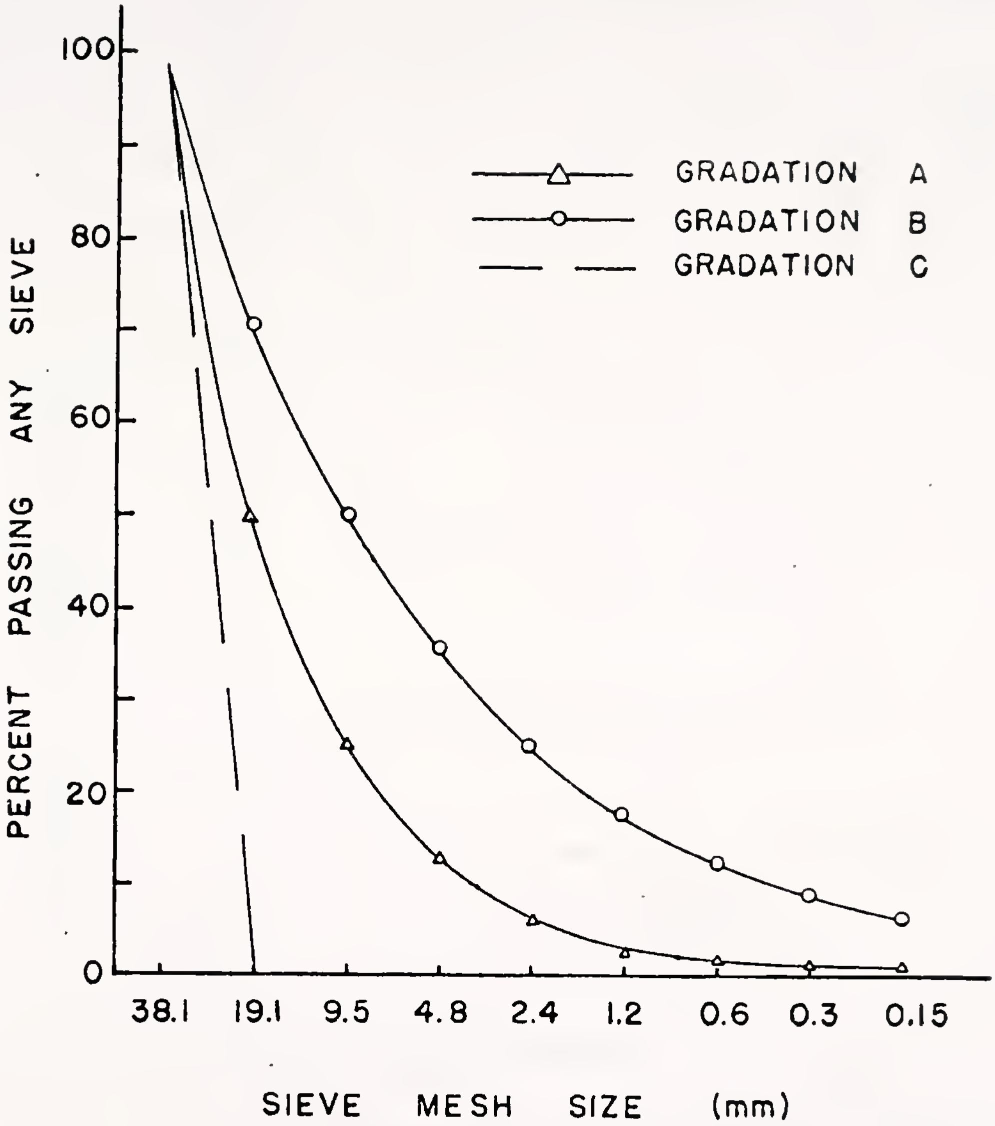


FIGURE 22 COMPARISON OF INITIAL GRADATIONS USED TO OBSERVE GRADATION EFFECTS

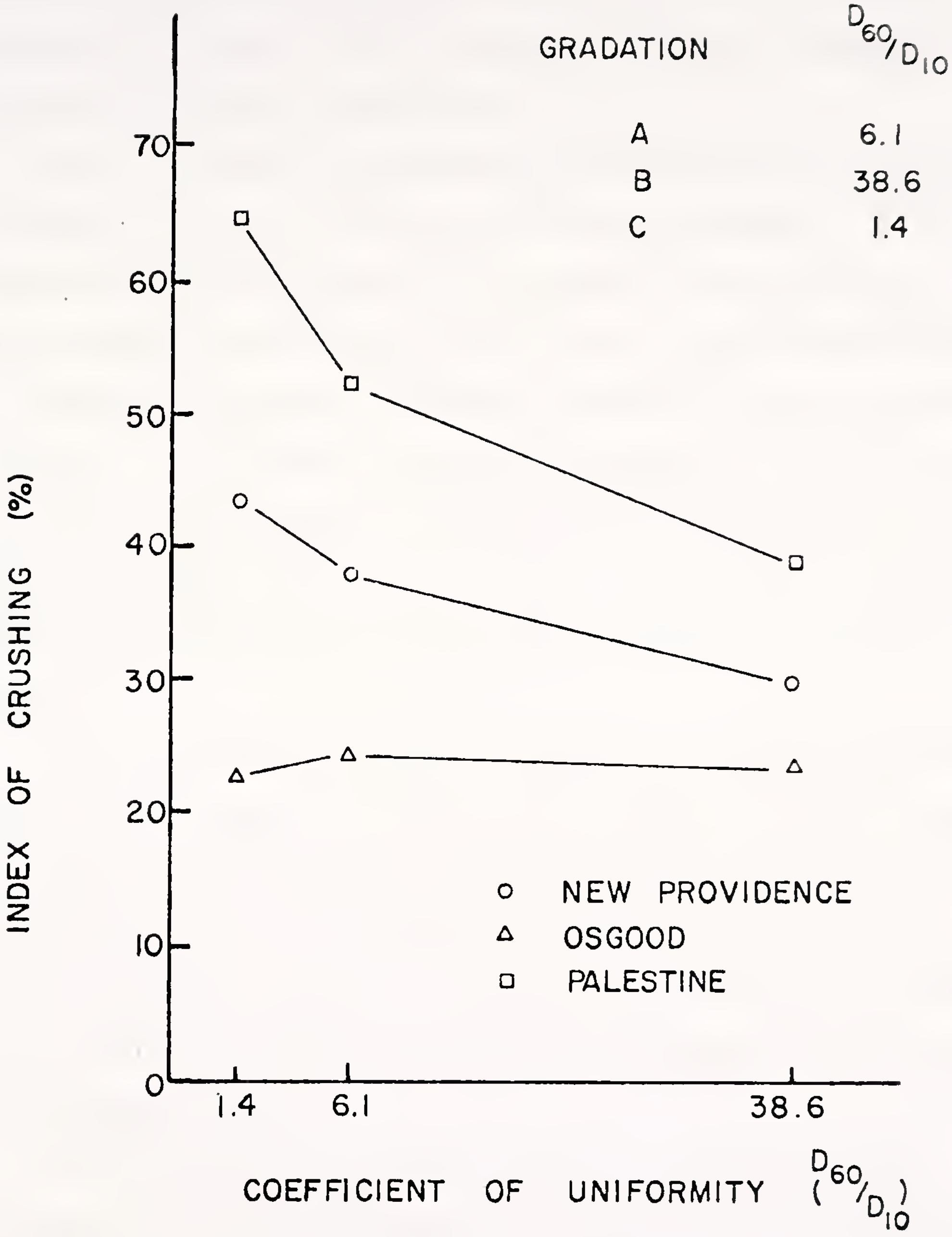


FIGURE 23 EFFECT OF GRADATION ON INDEX OF CRUSHING

corresponding respectively to 60 and 10 per cent passing on the cumulative distribution curve. The C_u increases as the initial gradation becomes more well-graded (ASTM D-2487).

The New Providence and Palestine shales experienced a decrease in degradation as the C_u increased for the initial gradation. The observed behavior may be explained by the presence of contact points within the sample. Fewer contact points exist in a narrow-banded gradation, causing the application of higher concentrated loads to individual aggregates. As illustrated in Figure 24, the absence of contact points also leaves larger portions of aggregates unsupported leading to greater bending and shearing stresses. The combined effect of larger loads and longer, unsupported distances on the aggregates results in greater degradation.

The number of contact points increases as the initial gradation becomes more well-graded. The distribution of compactive loads through the increased network of contact points reduces the loads concentrated at specific points. The increased contacts also provide more points of support for individual aggregates. The effect of increasing the number of contact points will lead to a reduction in degradation until little or no degradation occurs.

The initial gradation did not affect the degradation of the Osgood samples, although the number of contact points increased for the more well-graded samples. The Osgood shale was the hardest of the three test shales. Therefore, the compactive effort was probably insufficient to reflect any major effect by the initial gradation.

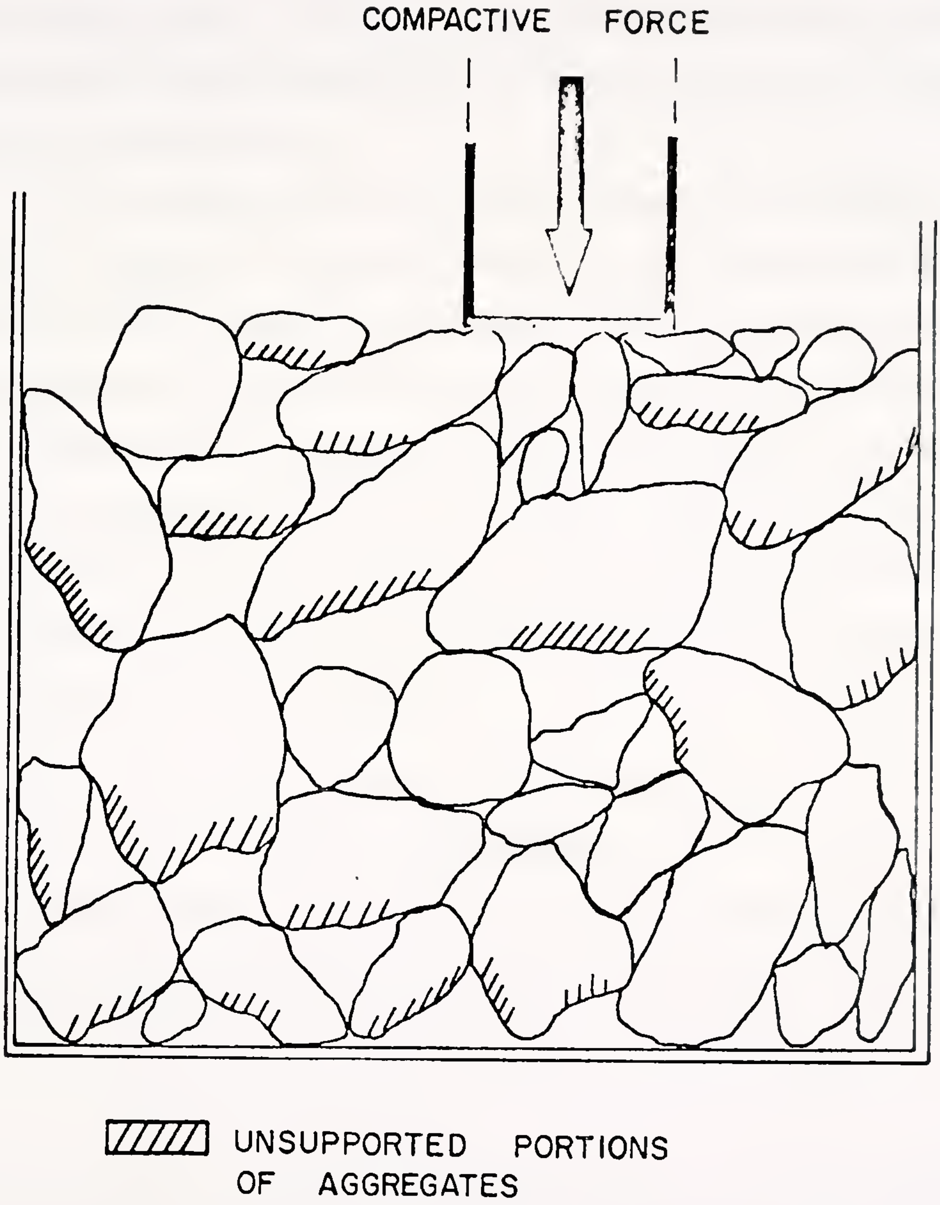


FIGURE 24 IDEALIZED REPRESENTATION OF UNSUPPORTED AGGREGATE PORTIONS

The effect of the initial gradation was much greater for the Palestine shale than for the New Providence shale and was negligible for the Osgood shale. This observation demonstrates that the relationship between grading and degradation is a function of both the shale type and compactive effort.

The dry density was also affected by the initial gradation. Figure 25 displays an increase in density for all three shales as the coefficient of uniformity for the samples increased. Medium and small size aggregates became available to fill the voids as the initial gradings changed from narrow-banded to well-graded. Degradation generally produces fragments which also fill the voids. However, as seen in the case of the Osgood shale, density may increase independently of the degradation if a sufficient distribution of medium and small aggregates is provided in the initial grading.

4.1.4 Procedure for Analyzing the Effects of Maximum Aggregate Size

Three gradations were used to evaluate the effect of the top aggregate size on degradation and density. The cumulative distribution curve for each of the gradations followed the equation;

$$P = (d/D)^1$$

where P is the percent passing any sieve size, d is the sieve mesh opening, and D is the top aggregate size. The gradation with a top size of 38.1 mm (1.5 in) was identical to gradation A used in developing the standard test. The subsequent gradations, A1 and A2, had top sizes of 19.1 mm (0.75 in) and 9.5 mm (0.38 in) respectively. The

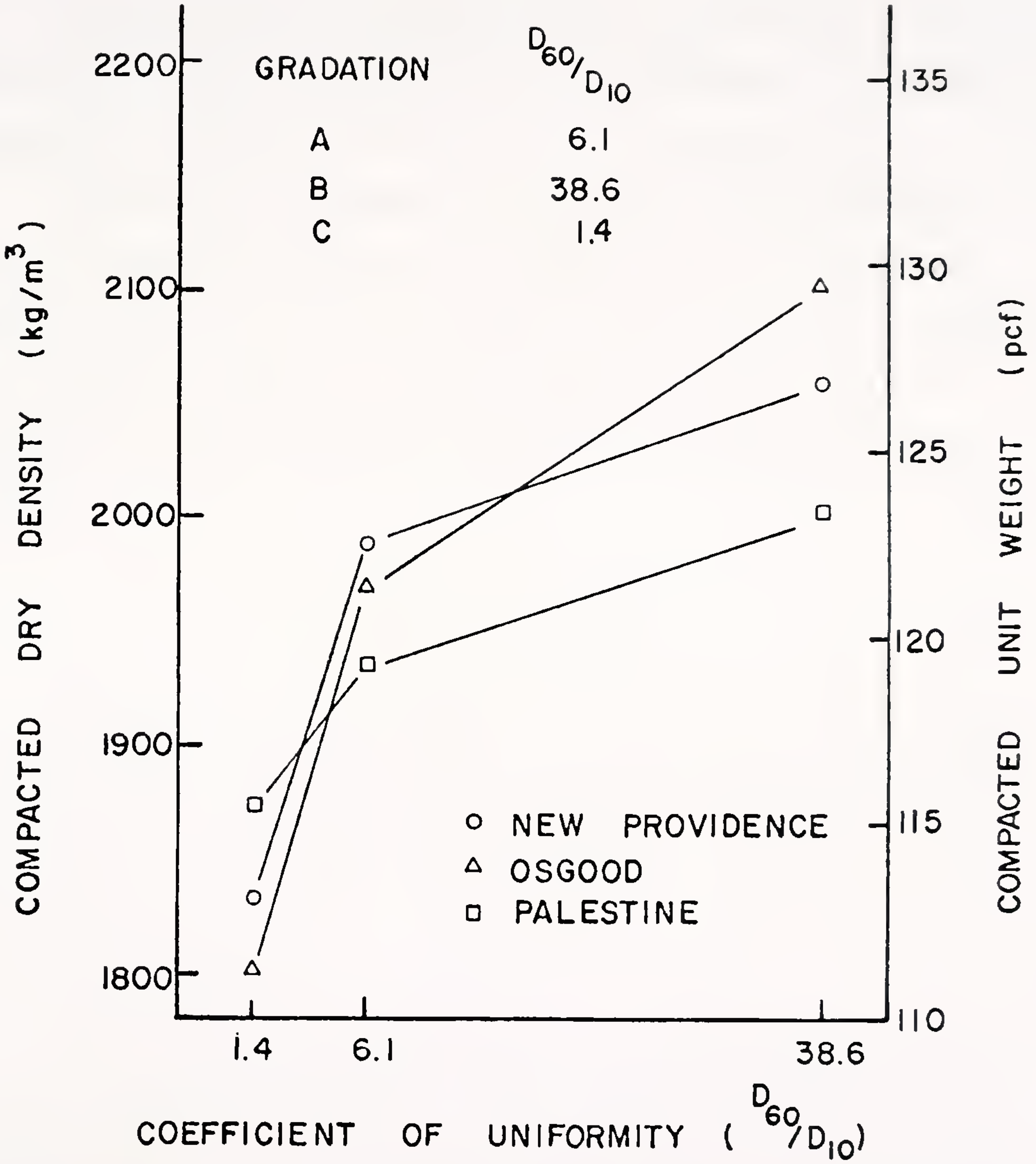


FIGURE 25 EFFECT OF GRADATION ON COMPACTED DRY DENSITY

cumulative distribution curves for the three gradations are shown in Figure 26.

One sample each of gradations A1 and A2 were blended for each shale type. Results from previous tests using gradation A were incorporated into the analysis of the effect of top aggregate size.

The shales were compacted at their natural moisture contents using the procedure developed for the standard test. The trimming, weighing, and dry sieving of the compacted samples followed the procedures discussed in Section 3.6.3 and Appendix C. Moisture content determinations were made for each sample for the calculation of dry density.

4.1.5 Results and Discussion of the Maximum Size Effects

The effect of the top aggregate size on degradation and density is demonstrated in Figures 27 and 28. The amount of degradation for all shales increased as the maximum size increased. The aggregate size distributions which illustrate the differences in degradation are given Figures A24 through A29.

This behavior can be attributed to two factors. First, the number of contact points available to carry the compactive loads decreased as the maximum size and the corresponding mean aggregate size increased. The reduction of contact points resulted in greater concentrated loads within the sample skeleton and led to increased crushing and fracturing.

Second, the probability of a structural weakness within an individual aggregate of fissile material increases as the aggregate size increases. Therefore, larger sizes are more likely to break down

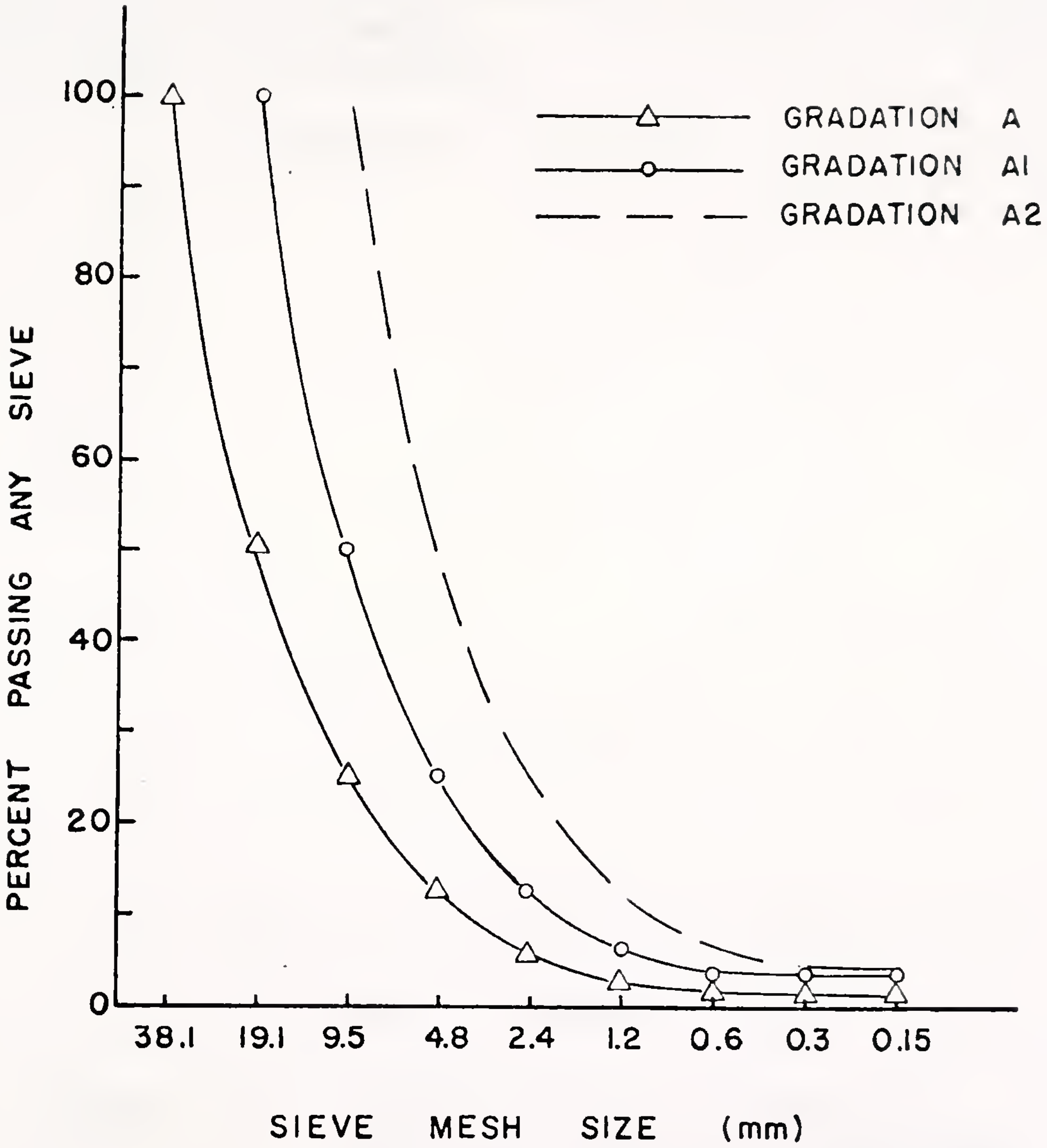


FIGURE 26 COMPARISON OF INITIAL GRADATIONS WITH VARIATION IN TOP AGGREGATE SIZE

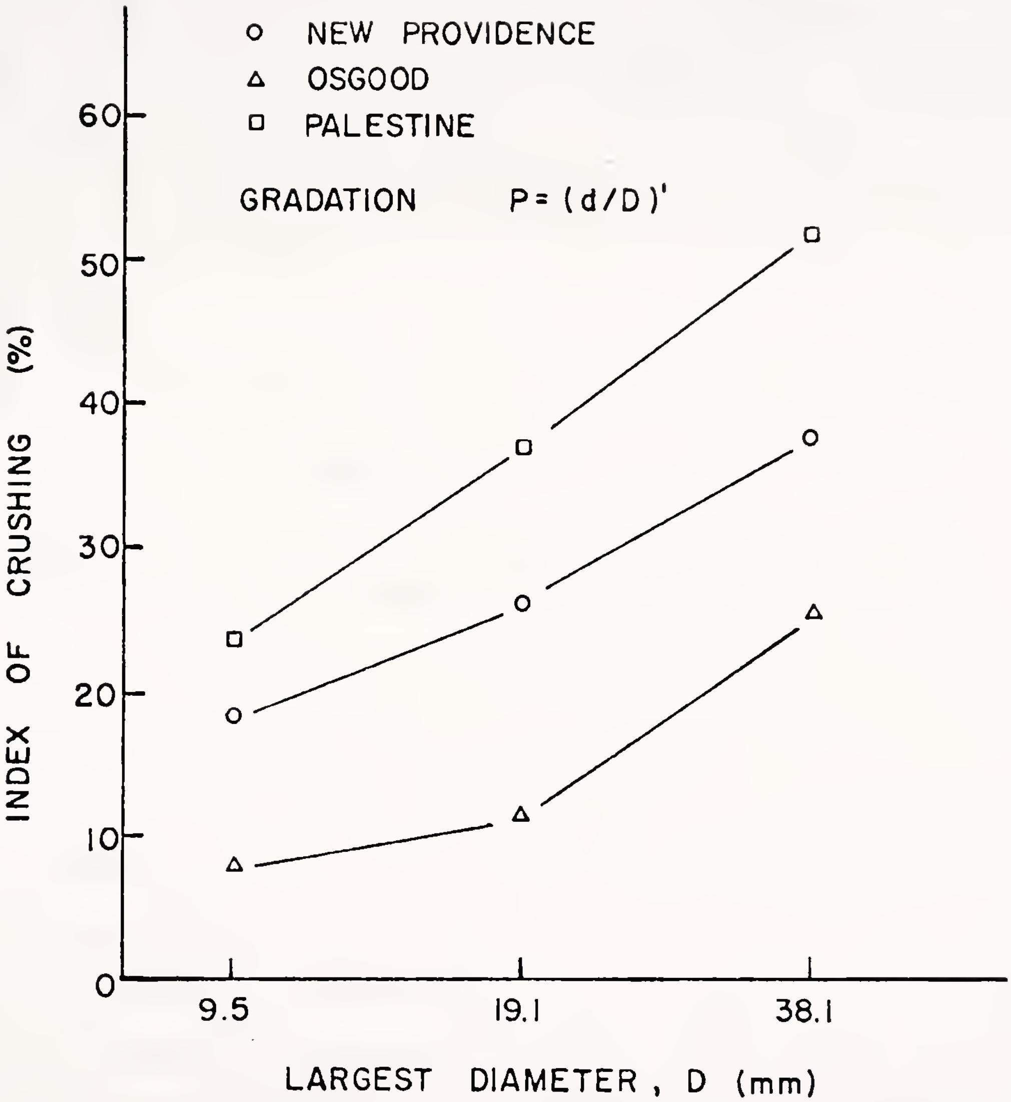


FIGURE 27 EFFECT OF TOP AGGREGATE SIZE ON INDEX OF CRUSHING

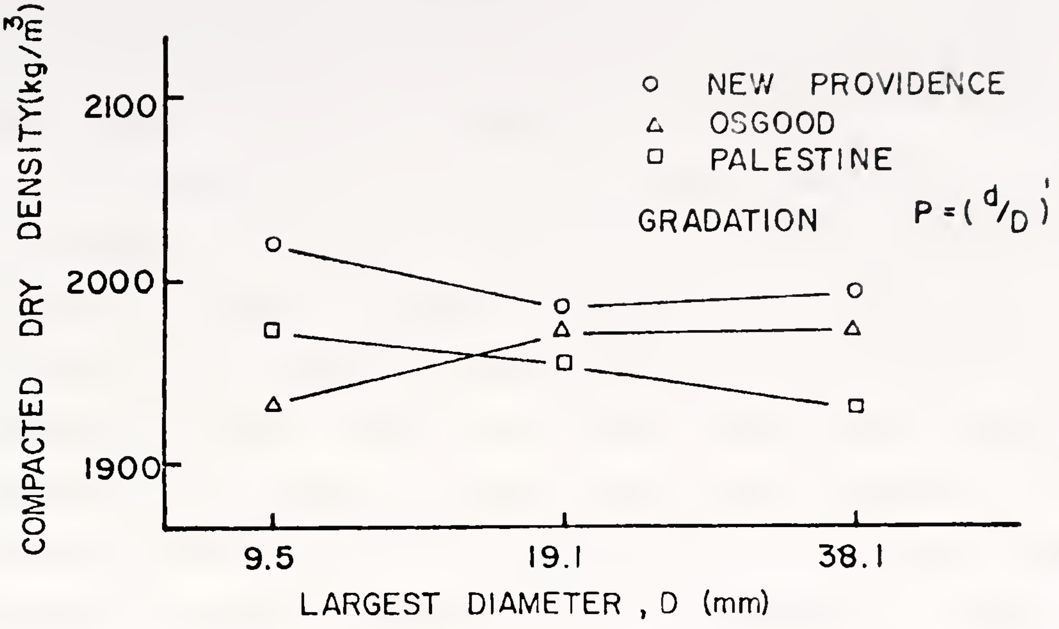


FIGURE 28 EFFECT OF TOP AGGREGATE SIZE ON COMPACTED DRY DENSITY

than smaller sizes under similar loading conditions.

The effect of the top size on the compacted dry density was not well established. The differences in the dry densities of the three gradations for each shale were small and demonstrated no general trend. The relationships of maximum size and unit weight established by Bailey (1976) indicated that the effect of maximum size was coupled with the level of compactive effort. In consideration of Bailey's results, the effect of maximum size on dry density for the samples in this testing sequence would have probably been more pronounced at a higher compactive effort.

4.1.6 Conclusions

Several conclusions may be drawn from the relationship between initial gradation, maximum size, degradation, and density.

In general, shales can be expected to degrade more under a given compactive effort than the stronger materials used in highway construction. However, the results from the analysis of initial gradation indicate that the basic degradation behavior of the shales at their natural moisture contents does not differ greatly from the general degradation trends observed for highway pavement mineral aggregates. In this context, previous experience with the degradation of highway materials can possibly be applied to the prediction of shale degradation behavior.

The results of samples compacted at various effort levels in the initial stages of the testing program established a parallel relationship between degradation and dry density. While such a relationship does exist, it is important to understand that the interaction of degradation and density is totally dependent on the compaction conditions. The analysis of initial gradation demonstrated that density may increase while the degradation decreases. The evaluation of maximum size presented evidence of increasing degradation which did not greatly influence the density. Therefore, any relationship between density and degradation is valid for only the corresponding set of compaction variables.

The application of the standard test to the study of initial gradation and maximum size served as an evaluation of the test itself. Two cases were found where the level of compactive effort used by the test was insufficient for a complete analysis of the compaction

variable. These cases were the influence of grading on the degradation of the Osgood shale and the influence of maximum size on dry density. The test could be adapted to these situations by increasing the compactive effort.

In general however, the test was successful in establishing the effects of initial gradation and maximum size on density and degradation. Further use of the standard test will provide additional opportunities to evaluate the testing process.

4.2 The Effect of Moisture on Degradation and Dry Density

4.2.1 Review

The application of water to aid the compaction of shale materials is a common construction practice. A compacted shale embankment may also become saturated at some point in its service life. An understanding of the effect of moisture on shale behavior is therefore essential for the successful use of shale as a construction material.

Research in rock mechanics has provided a vast amount of information concerning the effects of moisture on the compressive strength of rock specimens. Krynine and Judd (1957) summarized the results of several research programs by stating:

> "Saturation decreases the compressive strength of a rock. In one study the decrease in compressive strength because of a wet condition was 12 per cent for granite; in another from 4 to 8 per cent for granite and marble and from 10 to 20 per cent in layered sandstones, limestones, and slates. The higher the porosity--and thus the greater chance of saturation--the lower will be the strength of the rock when saturated. There is a definite relation between the percent sorption and compressive strength: As the per cent sorption increases, the compressive strength decreases."

The reduction in strength caused by moisture can be observed in forms of testing other than the unconfined compression test. Broch and Franklin (1972) used the point load strength test to demonstrate the strength reduction in various rock cores due to increasing saturation with water.

Investigations into the moisture effect on shales were prompted by the frequent collapses of the shale roofs and pillars utilized in coal mines. A study by Parker (1970) linked failures of the shale roofs in the White Pine mine with seasonal fluctuations in the shale moisture content. During the testing, he found that the strength of a moist shale was one-half of the dry shale strength.

Van Eeckhout (1976) also investigated the strength reduction due to moisture in coal mine shales. Specimens of shale were subjected to various degrees of humidity ranging from 0 to 100 per cent. The corresponding reduction in strength varied from 100 to 50 per cent of the air-dried value as the humidity increased.

Bailey (1976) used the point load strength test to evaluate the relationship between moisture content and shale strength. He allowed the moisture values to range from zero (oven-dried) to the natural moisture content of the shale. Bailey (1976) found several shales that displayed a linear decrease in strength as the moisture content increased. In addition to the moisture relationship, large variations in the strength values for the oven-dried samples were observed and attributed to the formation of cracks in the dried samples.

Increasing the moisture content of a shale by adding water will cause slaking in many cases. Slaking is loosely defined as "the

crumbling and disintegration of earth materials when exposed to air or moisture" (American Geological Institute, 1976).

The physical properties of a shale such as the mineralogy, porosity, and fissility will affect the amount of slaking that may occur. Surendra (1979) has found that the introduction of chemical additives to the slaking fluid can alter the amount of slaking for a given shale type.

Several theories have been presented to explain slaking in dried clays and shales. Terzaghi and Peck (1967) stated that the immersion of a dried clay specimen in water would saturate the outer portions of the sample and trap air in the inner portions. The inner air pressure would increase and produce a tensile failure of the specimen.

Chenevert (1970) proposed that slaking was a result of the swelling of expansive clay minerals when exposed to water. The explanation by Tschebotarioff (1973) was also based on swelling, but was not restricted to expansive clays. He stated that slaking was a surface phenomenon where outer layers exposed to water swelled and sloughed away from the specimen. The process was then repeated on the newly formed surfaces.

The work by Moriwaki (1975) on artificially sedimented clay samples indicated that slaking was caused by water removing the cementing agents.

The true cause of slaking is more likely a combination of the above explanations (Surendra, 1979). Whatever the cause, the breakdown of shale through slaking becomes an important consideration when moisture is added to shale.

Bailey (1976) used impact compaction tests to investigate the moisture-density relationship of shales. In general, increasing the moisture content resulted in an increased dry unit weight. A double-peaked curve was observed for some of the compaction tests but was not specifically investigated.

Kneading compaction tests by Abeyesekera (1978) on moistened but uncured samples of shale demonstrated increasing dry density with increasing amounts of added water for some of the samples. This trend, however, was not consistent and varied considerably with changes in compaction pressure, initial gradation, and initial aggregate moisture content.

Witsman (1979) performed impact compaction tests at three compactive efforts to establish the moisture-density relationships for moistened shale samples cured 24 hours. The results of the tests produced moisture-density curves similar to the Proctor curves for cohesive soils.

He then attempted to reproduce the curves with the use of the kneading compactor. However, no single value of compaction foot pressure modeled the curves from the impact tests. For this reason, Witsman varied the foot pressure to obtain densities matching the Proctor-type curves for a given moisture level.

4.2.2 Procedure for the Analysis of Moisture Effects

The previous discussions provide evidence that the use of water during compaction will increase the degradation of a shale through a reduction in strength and slaking, and possibly increase the density of

the sample. A series of tests were performed to evaluate the relationship between the moisture content, degradation, and dry density for the three test shales.

Samples of shales weighing 8 kg (17.6 lb) were blended to fit gradation A (see Figure 7). Distilled water was added while hand mixing the sample to approximately obtain the desired moisture level. The values of optimum moisture content established for the shales by the Indiana State Highway Commission on minus No. 4 material were not considered appropriate for samples meeting the "A" gradation. Therefore, the selected moisture levels ranged between the natural moisture content and 6% wet of the natural moisture level.

The moistened sample was stored in a sealed plastic bag and cured 24 hours at room temperature to allow a more even distribution of the moisture. The cured sample was then placed in a pan and mixed once again to avoid any aggregate segregation.

Adding water to the sample was expected to cause some breakdown of the shale through slaking. The reduction in strength resulting from the increased moisture content was expected to cause increased degradation of the shale during compaction. A procedure was developed to separate these components of the total amount of degradation.

Approximately 5 to 6 kg (11.0 to 13.2 lb) of the sample were compacted using the standard compaction procedure. The remaining material (approximately 2 to 3 kg (4.4 to 6.6 lb)) was weighed and wet sieved through a nest of sieves with mesh sizes of 19.1, 9.5, 4.8, 2.4, 1.2, 0.6, and 0.3 mm (U.S. Standard Sieves 3/4 in, 3/8 in, No. 4, 8, 16, 30, and 50).

The uncompacted portion of the sample was assumed to have a gradation very similar to the initial gradation. With this assumption, the change in gradation measured by the wet sieving would indicate the shale degradation that had resulted from the application of water, the sample mixing, and the actual sieving process.

Regular tap water was used for the wet sieving because the amount of material and lumping together of smaller aggregates required large quantities of water for complete washing. The minerals in the tap water were not expected to significantly alter the general trends of the shale behavior.

The material retained on each sieve was placed in a metal tin and dried at 105°C for 24 hours.

The compacted sample was trimmed and weighed. The trimmings and a small portion of the sample were used to evaluate the moisture content for both the compacted and the uncompacted shale.

The remainder of the compacted sample was then weighed, separated by hand, and wet sieved using the procedure previously described. Portions of the sample retained on each sieve were placed in metal tins and oven dried at 105°C for 24 hours.

The dry shale was weighed after the 24 hour period. The gradations of both the compacted and uncompacted materials were based on the estimated dry weights of the compacted and uncompacted portions of the sample.

The Index of Crushing (IC) for the uncompacted portion was taken to represent the degradation due to the sample preparation and wet sieving. The IC of the compacted portion represented the total

degradation of the testing process. The difference in the two IC values was assumed to be a measure of the degradation generated by the compaction.

4.2.3 Results and Discussions of Moisture Effects

Shales represent a broad transition between the rock and soil materials found on the earth. For this reason, the behavior of shales under different circumstances will also represent a transition between rock and soil behavior.

As discussed previously, shales experience a major reduction in strength with increasing moisture content and may slake when exposed directly to water. The effect of moisture on the durability of a shale will depend on where that particular shale lies along the transition between soil and rock. The test shales were selected from a class of nondurable shales and can therefore be expected to fully reflect the effects of moisture. However, each shale lies at a different point along the transition and will display a certain amount of unique behavior in the relationships between moisture, degradation, and dry density.

The effect of moisture on the total degradation (the Index of Crushing for the compacted portion of the sample) is illustrated in Figure 29.

The New Providence shale displayed a small increase in degradation which leveled off and eventually decreased at the highest moisture content. The Osgood shale demonstrated the largest increase in degradation with the application of water. However, the degradation became constant at the 6% moisture level and was not affected by further

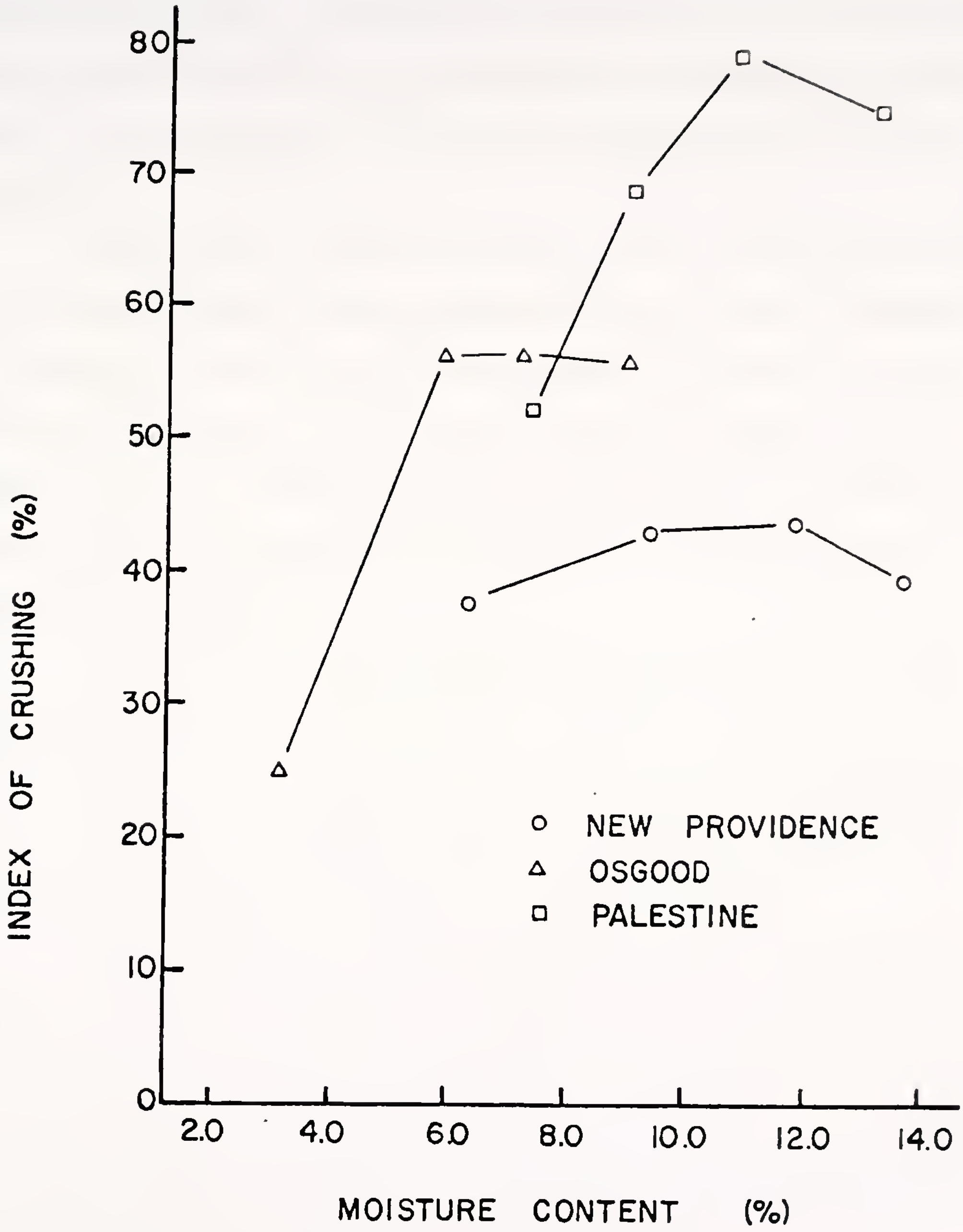


FIGURE 29 EFFECT OF MOISTURE CONTENT ON INDEX OF CRUSHING

increases in moisture. The Palestine shale degradation increased quickly with increasing moisture. The degradation experienced a well-defined peak at the 10.8% moisture level and then diminished with increasing moisture.

The attempt to measure the degradation due to the sample preparation and wet sieving produced results which were erratic and subject to uncertainty. The poor results were probably caused by the error induced by using only a small portion of the total sample. Such a situation could be corrected by using a larger amount of material in the initial sample and a more careful separation of the compacted and uncompacted portions.

The failure to establish the degradation caused by the sample preparation prevented the isolation of the various components affecting the total degradation. However, a measure of the combined degradation caused by slaking and increased aggregate breakage is still useful since both components will exist in the field conditions.

The relationship for moisture and dry density is displayed in Figure 30. The New Providence shale demonstrated a peak density value of 2004 kg/m^3 (123.6 pcf) at a moisture content of 9.3%. The Palestine shale demonstrated a peak density of 1986 kg/m^3 (122.5 pcf) at a moisture level of 10.8%. The Osgood however, continued to increase in density over the range of moisture contents tested.

Several theories have been presented to explain the characteristic moisture-density curve found for fine-grained soils. The theories are summarized by Hilf (1975) as:

1. Proctor's capillarity and lubrication theory

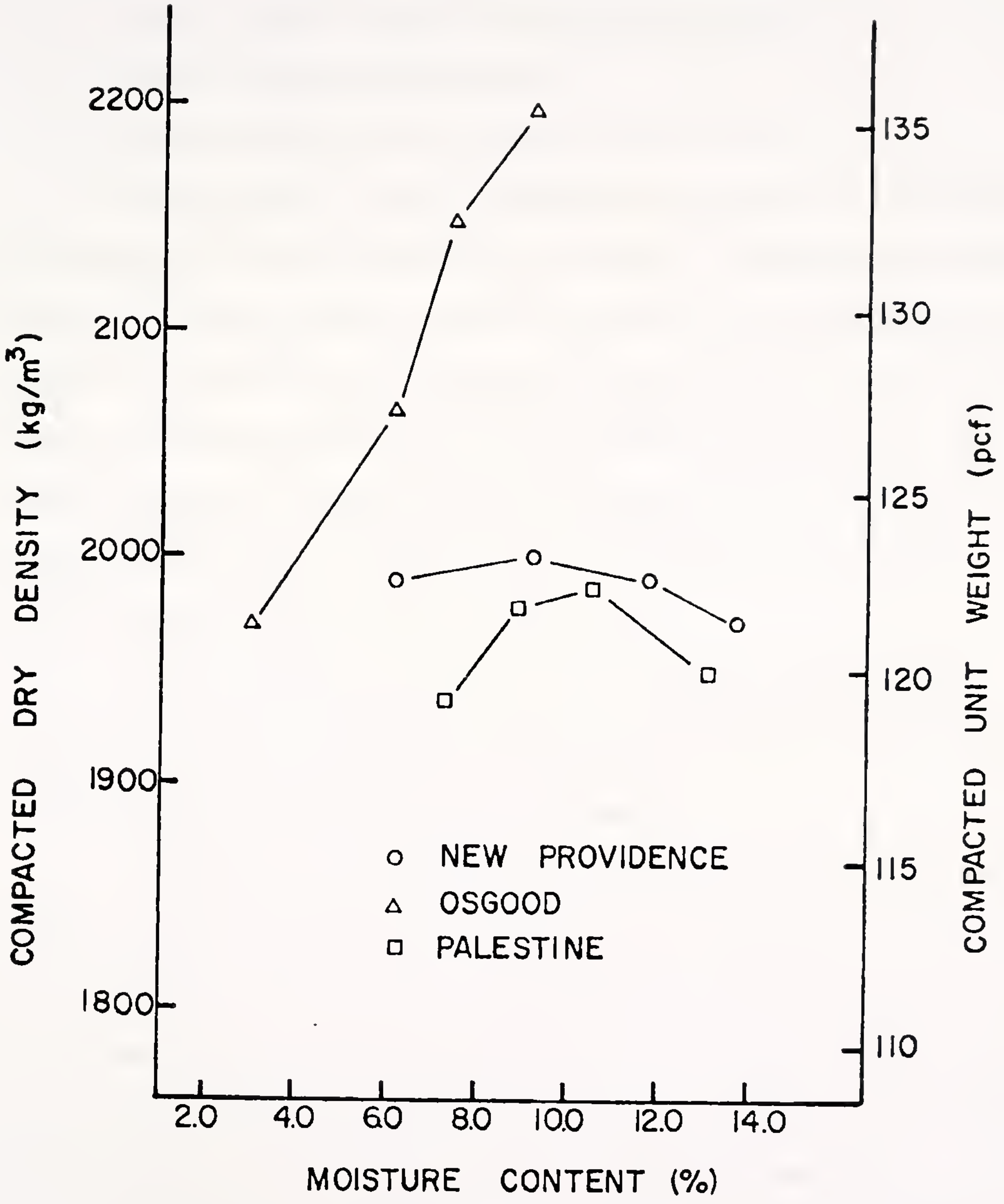


FIGURE 30 EFFECT OF MOISTURE CONTENT ON COMPACTED DRY DENSITY

2. Hogentogler's viscous water theory
3. Hilf's theory of capillary and pore air pressures
4. Lambe's surface chemicals and structural theory
5. Olson's effective stress theory
6. Barden and Sides' theory for macropeds of soil.

The theories by Proctor, Hogentogler, and Lambe all deal with the interaction of fine-grained soils and water. The theories by Hilf, Olson, and Barden and Sides basically state that increased density is caused by a reduction in the shearing strength between the soil particles. The theories based on shearing strength are in general agreement that the reduction in density after the optimum moisture content is reached is the result of discontinuous air channels which trap air voids and thus prevent further densification.

With the exception of the theory by Barden and Sides, the theories presented above approach compaction in terms of the movements and interactions of individual soil particles.

For shales, the primary units of concern are individual aggregates of various sizes which are composed of many fine-grained particles. The compaction characteristics of the sample will change as these aggregates break down; an effect not accounted for in the compaction theories. Also, the significance of the capillary forces or pore fluid pressures on the larger aggregates within a shale sample is uncertain.

Therefore, the theories may not be directly applied to the behavior of shales. However, portions of the theories may appear as the most logical explanation for the moisture effect on the compaction of shales as the aggregates break down into small domains.

Hodek (1972) presented a deformable aggregate model to explain the behavior of statically compacted kaolinite. The work by Garcia-Bengochea (1978) on pore size distribution for compacted clays provided evidence supporting the deformable aggregate model.

Additional work by Hodek and Lovell (1978) established a model to explain the moisture-density curve for compacted clays. In the model, the sample material is composed of soil aggregates of various sizes and moisture contents. For constant amounts of mixing water, the aggregate moisture content increases as the aggregate size increases. The gradation of these aggregates will play an important role in the compaction process by defining the sizes and distribution of the inter-aggregate voids.

The inter-aggregate voids are decreased during the compaction process by the rotation, translation, and deformation of the soil aggregates. The intra-aggregate voids are reduced to only a minor degree during compaction.

The plasticity of the soil aggregates increases with additional moisture, allowing further densification through deformation. However, further increases in density are not possible once the air channels within the mass become discontinuous.

The compaction model by Hodek and Lovell (1978) closely approximates the behavior of shales during compaction. The compaction properties of a shale sample will be influenced by the size and distribution of the aggregates. Also, the breakdown, translation, and rotation of the individual aggregates will increase the densification of the sample. Some amount of deformation may occur although the plasticity of the shale aggregates will not be as great as for the soil aggregates.

One important concept must be considered throughout the discussion on the moisture effects. The moisture content of a graded shale material cannot be compared directly with the moisture content of a fine-grained soil. Bailey (1976) found that the moisture contents for the individual size groups of a soaked gradation ranged from 10% to 60% as the aggregate size decreased from 19.1 mm (0.75 in) to 0.3 mm (0.01 in). This is not surprising due to the larger surface area and thus greater absorption and adsorption capacity for the finer pieces. Also, the moisture gradient from the outer surface to the center of an aggregate will depend on the aggregate size, porosity, and pore size distribution.

Therefore, the moisture value given for a particular sample is at best an average value for all the individual aggregates in the sample.

Independent discussions of the degradation and dry density are not effective in evaluating the moisture effect since several factors exist which closely link the two characteristics. However, the differences in the materials warrant an individual analysis of each shale type.

The Osgood shale demonstrated a substantial increase in degradation when water was added. The corresponding increase in density could be related to the greater number of medium and small size aggregates created by the slaking and increased breakage of the shale since a major effect of capillary or pore water pressures on the predominately coarse-grained shale samples seems unlikely.

The degradation of the Osgood shale remained constant with increasing moisture content above the 6% level. This can be explained by the limited amount of water the shale could absorb in a 24 hour period. Any additional water in the sample essentially did not contribute to the strength reduction or slaking of the shale.

The dry density however, continued to increase with additional moisture. Such behavior indicates that factors other than slaking and additional degradation were affecting the density.

For the Osgood shale, the water that was not absorbed into the aggregates remained on the surface of the individual pieces. This adsorbed water created a moist, clay film around each piece and served as an intergranular lubricant. A much higher state of packing was obtained by reducing the frictional forces between the aggregates. Additional water at the highest moisture level was available to create a clay film on the unmoistened surfaces of the fragments produced by the fracturing of the large pieces. The ability to lubricate the increased number of aggregates formed by the degradation process resulted in even higher dry density values.

The Palestine shale was the softest and most porous of the test shales. Most of the water added to increase the moisture content was absorbed by the shale. Excessive moisture appeared as adsorbed water on the aggregates only at the highest moisture level.

The degradation of the Palestine shale increased substantially with each increase in moisture content up to the 10.8% level. The corresponding increases in dry density were primarily a direct result of the more dense packing made possible by the formation of medium and

small size aggregates. The absence of excessive water on the surfaces of the aggregates would have limited the effect of intergranular lubrication among the large and medium size pieces. However, the presence of water within the sample may have provided an important amount of lubrication for the fine aggregates.

At the highest moisture level (14.5%), excess water was observed on the aggregates. During compaction, the sample began to heave with every drop of the rammer and displayed a decrease in dry density. Similar behavior is found in cohesive soils very wet of optimum and is presently associated with trapped air voids within the sample.

Two interesting points can be developed from the assumption that the behavior of the wet Palestine shale during compaction was the result of occluded air voids. First, the trapped air voids would actually carry some of the compactive loads and in effect, cushion the aggregates. This would explain the reduced degradation observed for the Palestine shale at the 13.1% moisture level.

Second, the Palestine shale at the highest moisture level behaved very much like a cohesive soil. This would indicate that the shale had moved closer to a soil-like material from its original position along the soil-rock transition.

The New Providence shale was classified as the most durable of the test shales based on the results of the two cycle slake durability index reported by the Indiana State Highway Commission. The addition of water to the New Providence shale increased both the degradation and density although not to the extent observed for the Palestine shale. The degradation and density began to decrease as the moisture content increased further.

The physical features of increased degradation, intergranular lubrication, and occluded air voids used to explain the compaction behavior of the Palestine shale existed for the New Providence shale as well. However the greater durability of the New Providence shale limited the effect of these moisture-related features.

4.2.4 Conclusions

Several conclusions may be drawn from the results and discussion of the moisture effect on shales. The conclusions are summarized below.

1. An increase in moisture content will increase the total degradation of a shale up to a limited value for a given compactive effort.
2. Control of the shale moisture content by thorough mixing with water and adequate cure time will increase the density up to a limited value for a given compactive effort.
3. The extent of the moisture effect on a shale will depend on the physical properties of the shale such as the magnitude and distribution of porosity, the mineralogical composition, and the durability.
4. A shale represents a material intermediate between rock and soil, and can approach a more soil-like material during the compaction process with increasing moisture content. The amount of change a shale will experience during compaction is a function of the shale properties and the moisture.

The use of water to aid field compaction must be approached carefully due to the problems encountered with large aggregates,

inadequate mixing, and inadequate curing. However, the results of the analysis on the relationships of moisture, degradation, and dry density may still be useful in understanding and improving the field compaction of shales.

5. TESTS TO PREDICT SHALE DEGRADABILITY

5.1 The Use of the Standard Impact Test as a Measure of Shale Degradability

5.1.1 Introduction

The hard but nondurable shales present special problems in compacted shale embankments. The large pieces which are not degraded during compaction can lead to the presence of large voids in the fill. During the wetting and drying cycles of the embankment, the shale pieces disintergrate and collapse into the voids. According to Wood et al. (1978) the resulting disruption in drainage accelerates fill settlements.

Construction of embankments with nondurable shales generally requires special efforts to thoroughly degrade the material in an attempt to minimize the potential hazards. A laboratory test which could predict the degradability of a shale would be helpful in assessing the amount of extra effort and the type of compactive equipment that would be required during construction.

The standard impact test developed from this testing program could serve as an index test to measure the degradability of a shale.

5.1.2 Procedure and Results

The results from the impact compaction tests for the initial testing sequence were compared for the 790 kN-m/m^3 (16,500 ft-lb/ft^3)

effort level. The tests performed at this level demonstrated sufficient repeatability and avoided the problems of sample separation associated with the higher effort levels. The samples were all blended to fit gradation A and were tested at their natural moisture contents.

The average Index of Crushing (IC) for each shale is given in Table 6.

Table 6. Average Index of Crushing (IC)

SHALE	NEW PROVIDENCE	OSGOOD	PALESTINE
IC (%)	37.6	25.1	51.8

The representative aggregate size distribution for each shale is illustrated in Figure 31.

5.1.3 Discussion

A successful index test based on the degradability of shales must be able to differentiate among various shales. The standard compaction procedure developed in this testing program demonstrated such a capability in both the Index of Crushing and the aggregate size distributions for the three test shales.

The effects of compactive effort, initial gradation, and maximum size have been discussed in previous sections. It was demonstrated that a change in any one of these variables would change the degradation. Therefore, the sample and compaction variables must be held constant for

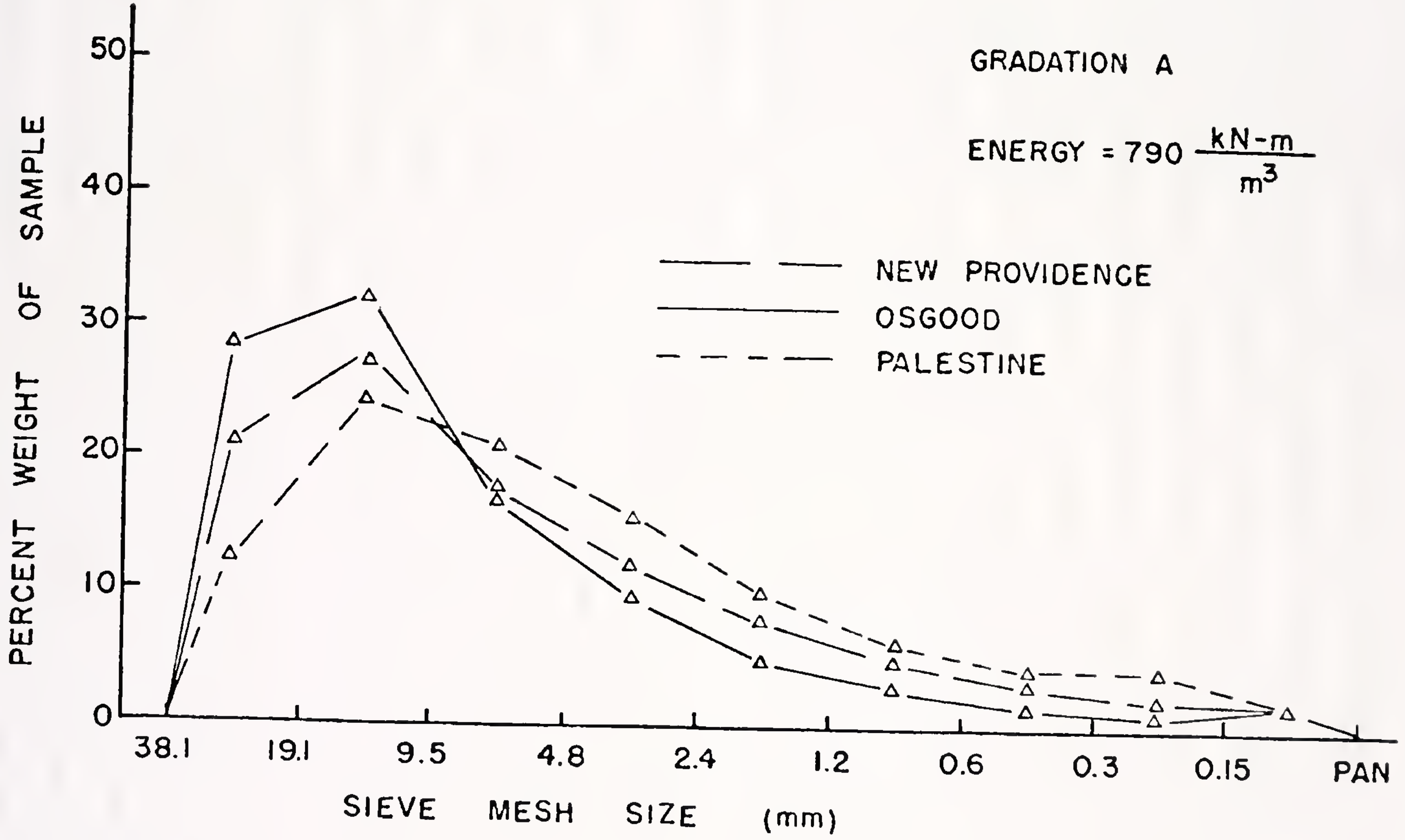


FIGURE 31 AGGREGATE SIZE DISTRIBUTIONS FOR NEW PROVIDENCE, OSGOOD, AND PALESTINE SHALE; IMPACT COMPACTION

an index test which would be used to measure degradability and eventually to classify shales.

The 790 kN-m/m^3 (16,500 ft-lb/ft^3) effort level is reliable and should be retained for the degradability index test. The "A" gradation with its exponential cumulative gradation curve and 38.1 mm (1.5 in) top size, provides a reasonable model for the gradations which may be encountered during the construction of a compacted shale embankment.

However, the entire "A" gradation is not completely necessary. Compaction tests were performed on samples containing only the top four size groups of gradation A (38.1 - 19.1 mm (1 1/2 - 3/4 in), 19.1 - 9.52 mm (3/4 - 3/8 in), 9.52 - 4.76 mm (3/8 - No. 4 sieve), 4.76 - 2.38 mm (No. 4 - No. 8 sieve)). The amount of shale used for each of the four size groups was identical to the amount used for the corresponding size groups in the entire "A" gradation.

The results from the compaction tests using samples with partial gradations are given in Table 7. The Index of Crushing (IC) results for the partial gradation samples can not be directly compared with the IC values in Table 6 due to the differences in moisture content and initial gradation among the samples. However, the results in Table 7 indicate that the partial gradation may be successfully used in a test to measure shale degradability.

Using the partial "A" gradation simplifies the testing procedure. The use of only the four top sizes also emphasizes the role of the larger shale pieces in the degradation process and may therefore lead to a more distinct classification among shales. For these reasons, the partial "A" gradation is recommended when using the standard impact test to classify the degradability of a shale.

Table 7. Index of Crushing (IC) for Samples with Partial "A" Gradation.

SHALE	NEW PROVIDENCE	OSGOOD	PALESTINE
IC (%)	37.6	14.2	54.7

Moisture will also affect the degradability of a shale. The testing of all shales at a standard moisture level would provide a more accurate method of comparing the materials. However, a standard moisture level is difficult to establish for shales. Drying all shales to zero moisture content would establish such a moisture standard. However, Bailey (1976) reported the formation of cracks in samples oven dried for the point load strength test. Large variations in the strength index were attributed in part to these cracks. Based on these observations, the formation of cracks in the aggregates during the drying process could produce large variations in the degradability of a sample. Also, the degradability of a shale at zero moisture content would be difficult to correlate with the degradability of the same material at its natural moisture content during field compaction.

Complete saturation could also be considered as a standard moisture. However, the serious effects of moisture on shales (slaking and strength reduction) eliminate this alternative.

The natural moisture level appears to be the most realistic standard. Although the natural moisture contents will vary somewhat among the shales, this level will allow the classification of shales for their in-situ moisture conditions. By using the natural moisture

level, correlations between the index test and field degradation should be easier and the problems of moisture manipulation can be avoided.

5.1.4 Summary

The successful acceptance of an index test requires a great deal of experience and field correlation. The standard variables suggested for the degradation test may be altered to bring the test and field observations closer together. The test suggested by this program should therefore be considered the first model and not the finished product for the degradation index test.

5.2 The Use of the Point Load Strength Test in Predicting Shale Degradability

5.2.1 History of the Point Load Strength Test

The uniaxial and triaxial compression tests are widely accepted methods of determining the strength of rock. However, the sample preparation necessary to obtain the required specimen size and shape generally limits the use of the test to the stronger rock materials which can withstand the preparation techniques and are not broken down by water. The sides of the finished specimens must also be free of abrupt irregularities (ASTM D2938-71).

The specimen requirements greatly restrict the use of conventional compression tests for shales and other weak rocks. To overcome this problem, Hobbs (1964) extended the initial work by Protodyakov and Voblikov and developed a compression test for irregular lumps of rock. The test required no special sample preparation and was performed by crushing the specimen between two loading platens. A piece of carbon

paper and a piece of graph paper were placed between the specimen and each platen to measure the contact area.

The test developed by Hobbs (1964) was successful in estimating the compressive strength of weak rock, but the process of measuring the contact area was time consuming. Broch and Franklin (1972) suggested the use of a point load to simplify the testing of irregular specimens.

The use of a point load had previously been established as an indirect tensile test for cylinders and disks. An experimental study by Reichmuth (1968) used the point load test to establish a basic expression for tensile strength. The expression included a shape factor to provide a better correlation between the different sizes and shapes of specimens. The expression also included a factor for material brittleness.

According to Broch and Franklin (1972), other researchers used the point load testing of cores to establish correlations with blasting properties, drilling rates, and weakness planes in rock. The work by Hiramatsu and Oka (1966) established a mathematical model for the state of stress in specimens under point loading.

The testing procedure developed by Broch and Franklin combined the aspects of point loading and the use of irregular specimens. They found that the specimen size and shape effects were more severe for irregular lumps than for cores or disks, thus making the test less precise for those conditions. Broch and Franklin (1972) concluded that despite the reduced accuracy, the point load strength (PLS) test on irregular lumps was a suitable method for strength classifications of rocks.

The point load index (I_s) developed by Broch and Franklin (1972) was based on the formulation $I_s = P/D^2$ where P is the failure load and D is the initial distance between the loading points. The index and testing procedure were adopted by the International Society of Rock Mechanics in 1972. Additional work by Guidicini et al. (1973) and Bieniawski (1975) added support to the application of the point load index in rock mechanics, mining, and engineering geology.

The large variations in strength resulting from the shape and size effects of irregular lumps often limited the point load strength (PLS) test to cores or disks. Brook (1977) suggested a method of overcoming size and shape effects when he observed that the log - log plot of the failure load vs. the sample fracture area gave a linear relationship. Brook's strength index was based on the PLS value for a standard area.

5.2.2 Features of the Point Load Strength Test

The point load strength (PLS) test has several advantages. Irregular specimens which do not require special preparation may be used. The test may be performed quickly and inexpensively. The test is simple enough that portable testing equipment may be successfully developed.

The combined features of the PLS test are especially suited for testing shales. The elimination of special sample preparation allows the testing of weak or thinly laminated shales. The time saved by avoiding sample preparation, along with the short duration of the actual test, makes the testing of a large number of samples feasible. This capability is important considering the natural variations found in shales. The short time required to test an individual specimen also

reduces the changes in sample moisture content which may occur during testing. The use of portable equipment allows the testing of shales during the subsurface investigation programs and at the construction site.

The basic failure mechanism in the PLS test is the development of tensile stresses normal to the loaded diameter. A zone of compressive stresses exists near each loading point resulting in localized crushing. The crushing is a material characteristic which is automatically included in the test and will not significantly affect the results as long as the size of the loading points remains constant. The "point" load is distributed over a slightly greater area as the amount of crushing increases. The increase in area is not significant since the critical failure zone initiates in the center of the specimen and is not greatly affected by the surface conditions. (Bailey, 1976)

5.2.3 Special Problems When Testing Shales

The physical characteristics of shales which cause problems in the construction of embankments also create problems in the point load strength (PLS) test. Sample moisture content is one such problem. Previous work by Bailey (1976) shown in Figure 32 indicated that over a narrow range of moisture contents, shale strength (as measured by the PLS test) increased as the moisture decreased from the natural moisture content to the oven dry state. An increase of moisture was impractical due to the slaking of shale samples. Conversely, Bailey found that drying out of samples resulted in the development of weakening cracks. These problems favor the evaluation of the PLS index at the natural moisture content.

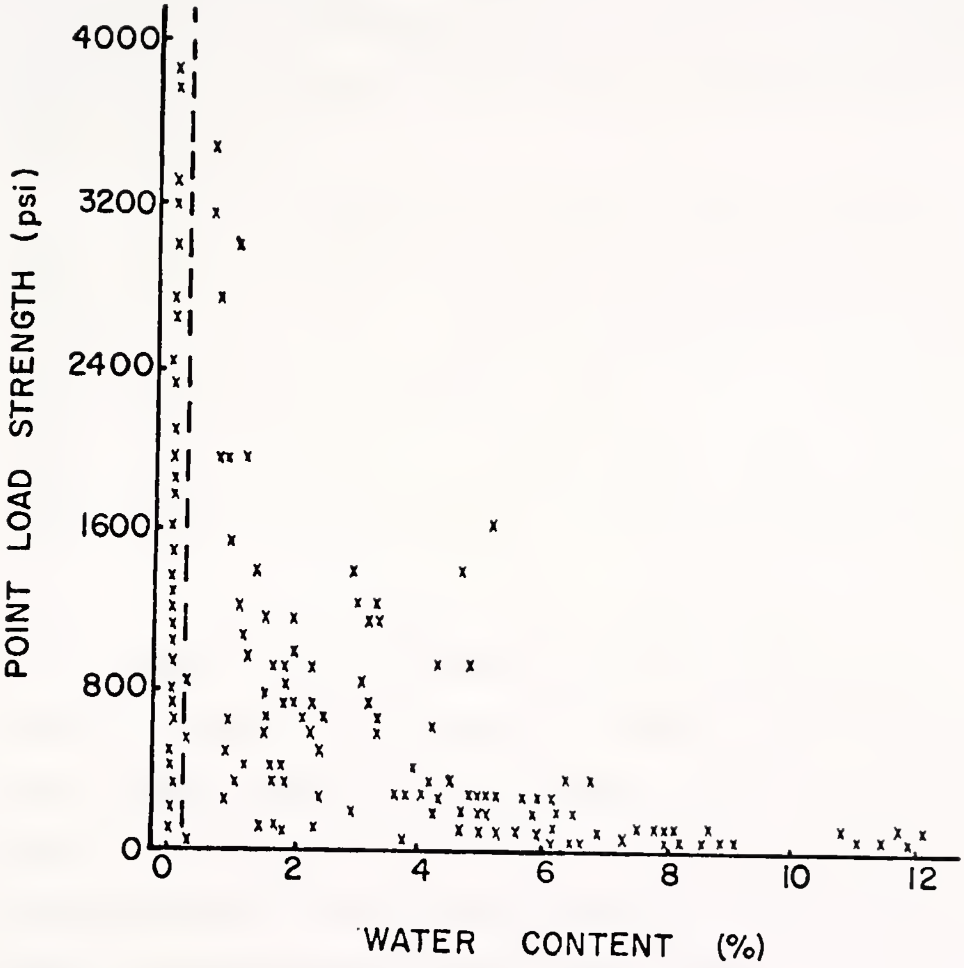


FIGURE 32 VARIATION IN POINT LOAD STRENGTH WITH WATER CONTENT FOR ALL SHALES TESTED
(AFTER BAILEY, 1976)

Shales exhibit stratification and bedding planes. The type of stratification and the thickness of the bedding planes affect both the fissility of the shale and the strength characteristics during the PLS test. Based on the sedimentary structure, shales may display spalling or even a form of successive failure as thin beds near the loading points fail individually. For such shales, the definition of failure load is arbitrary.

Samples for the PLS test require no special preparation. However, experience has shown that sample size has a major effect on the PLS index (Brook, 1977). The pieces selected for testing should be roughly equidimensional (Broch and Franklin, 1972). Since the fissility of a shale tends to produce flaggy or platey, rather than bulky, pieces, obtaining samples of uniform size becomes difficult.

5.2.4 Size Effect in Testing Shales

Although the size effect in the point load strength test has been well-documented for a variety of rock materials, no definite explanation has been established. Most current theories relate the size effect to the state of stress within a sample and the failure mechanism of point loading. Other researchers have noted that the size effect varies for different rock types (Franklin, 1979).

To evaluate the size effect in shales, approximately 150 samples were prepared from the New Providence and Attica shales of Indiana for point load strength (PLS) testing. The samples varied in thickness from 5.0 mm (0.20 in) to 38.0 mm (1.5 in). The data from the tests were grouped into size ranges of 6.4 mm (0.25 in) and are presented in

Table 8. As shown in Figure 33, the mean value of the PLS index from each size group decreased as the sample size increased.

5.2.5 Laboratory Testing

Samples of the test shales (New Providence, Osgood, and Palestine) were prepared for point load strength (PLS) testing. The results of the size effect study had indicated that the variation in the PLS index reduced as the sample size increased. Also, the higher failure loads associated with larger samples could be more accurately monitored from the loading device. For these reasons, the samples were prepared as large and uniform as possible.

The PLS test apparatus used in the study is shown in Figures 34 and 35. The lower platen and two guide bars are supported by a rigid base. The upper platen is fastened to a guide plate which slides freely along the guide bars. The platens are spherically tipped cones made of case hardened steel. The dimensions of the platens conform to the standards of the International Society for Rock Mechanics (1972).

The entire PLS apparatus was placed between the platens of a compression testing machine for loading. Samples with apparent bedding were placed in the PLS apparatus so that the load would be applied perpendicularly to the bedding planes. This was the direction of greatest strength for such shale types. After placing a sample between the points, a small seating load was applied, and an initial reading of the apparatus dial gage was used to calculate the sample thickness. The samples were tested at their natural moisture contents and subjected to a loading rate of 0.03 cm/min (0.01 in/min). Both load and sample deformation were monitored at regular time intervals using readings

Table 8. Size Effect on Point Load Strength Values

SHALE	SIZE RANGE mm (in.)	NUMBER OF SAMPLES	PLS VALUES		
			MEAN VALUE MPa (psi)	STANDARD DEVIATION MPa (psi)	COEFFICIENT VARIATION OF (%)
NEW PROVIDENCE	5.0 TO 12.7 (0.20 TO 0.50)	25	18.79 (2724)	6.59 (956)	35
	12.8 TO 19.0 (0.51 TO 0.75)	29	8.81 (1277)	3.55 (515)	40
	19.1 TO 25.4 (0.76 TO 1.00)	14	4.17 (604)	1.77 (256)	42
	25.5 TO 31.8 (1.01 TO 1.25)	5	3.03 (439)	1.28 (185)	42
	31.9 TO 38.1 (1.26 TO 1.50)	7	1.81 (262)	0.54 (79)	30
ATTICA	6.3 TO 12.7 (0.25 TO 0.50)	12	8.21 (1190)	2.54 (368)	31
	12.8 TO 19.0 (0.51 TO 0.75)	22	4.74 (687)	2.10 (305)	44
	19.1 TO 25.4 (0.76 TO 1.00)	19	2.82 (409)	0.88 (127)	31
	25.5 TO 31.8 (1.01 TO 1.25)	9	1.95 (283)	0.39 (56)	20
	31.9 TO 38.1 (1.26 TO 1.50)	5	1.43 (207)	0.41 (59)	29

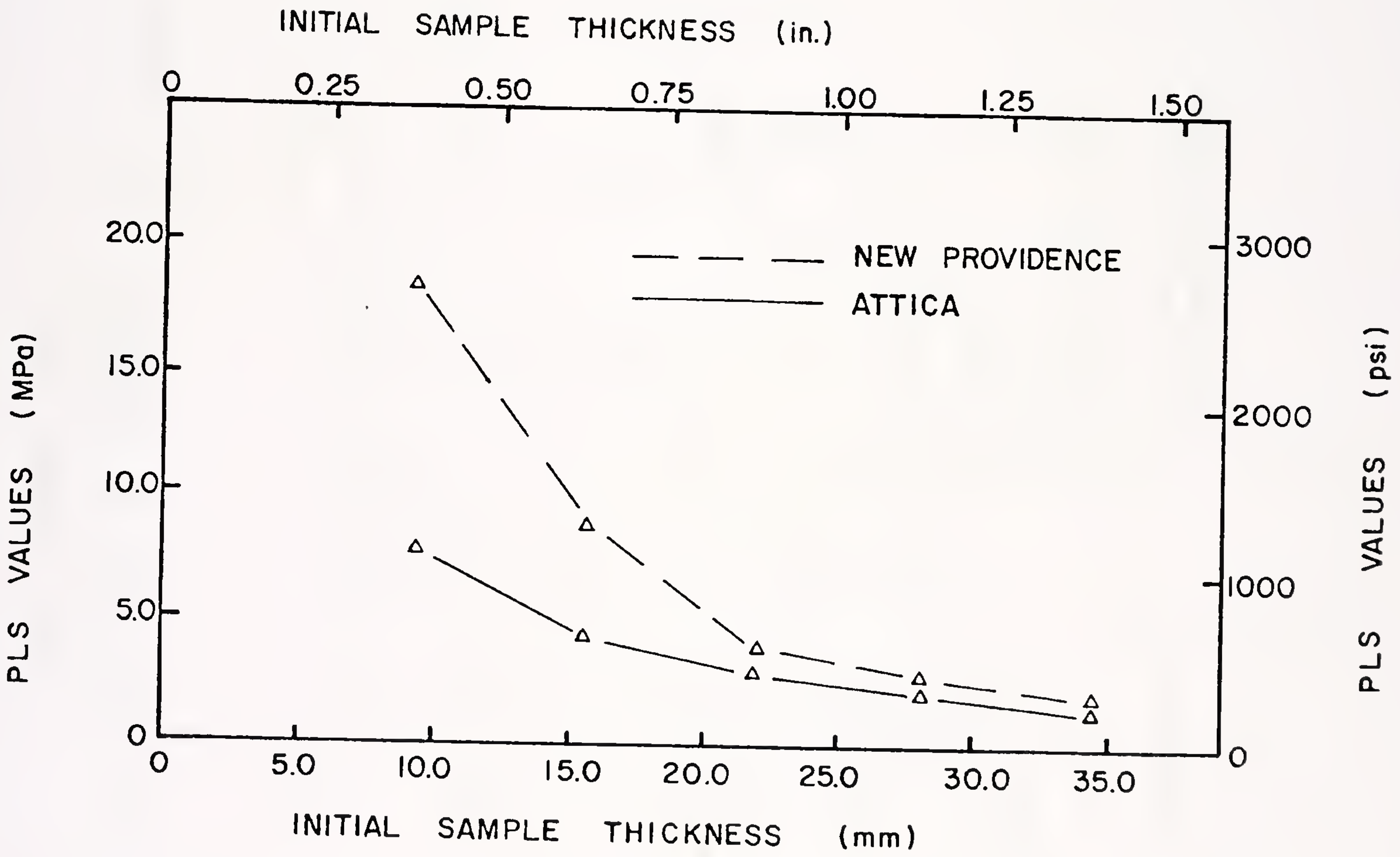


FIGURE 33 SIZE EFFECT ON POINT LOAD STRENGTH VALUES

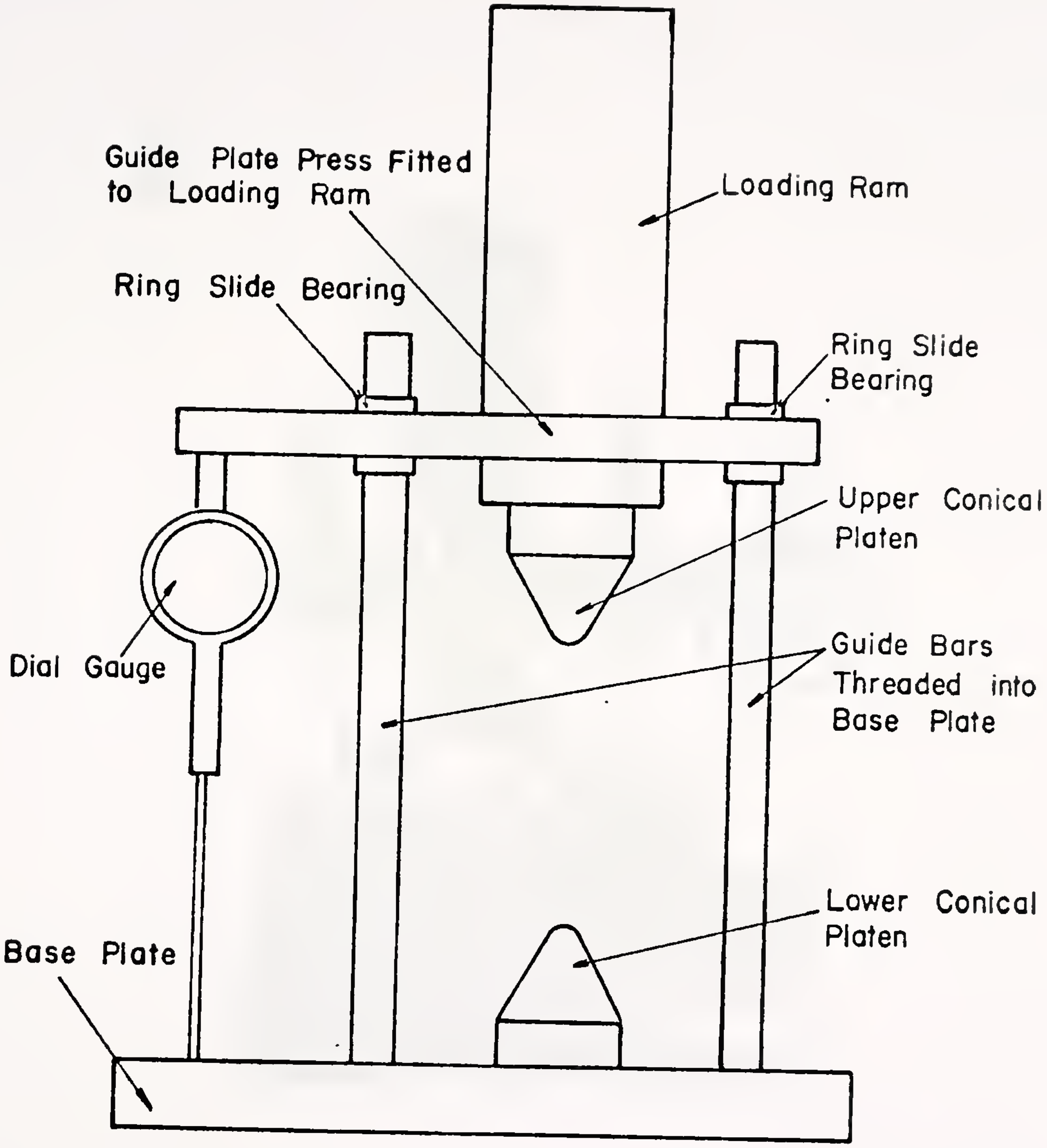


FIGURE 34 DIAGRAM OF POINT LOAD TEST APPARATUS

(AFTER BAILEY, 1976)

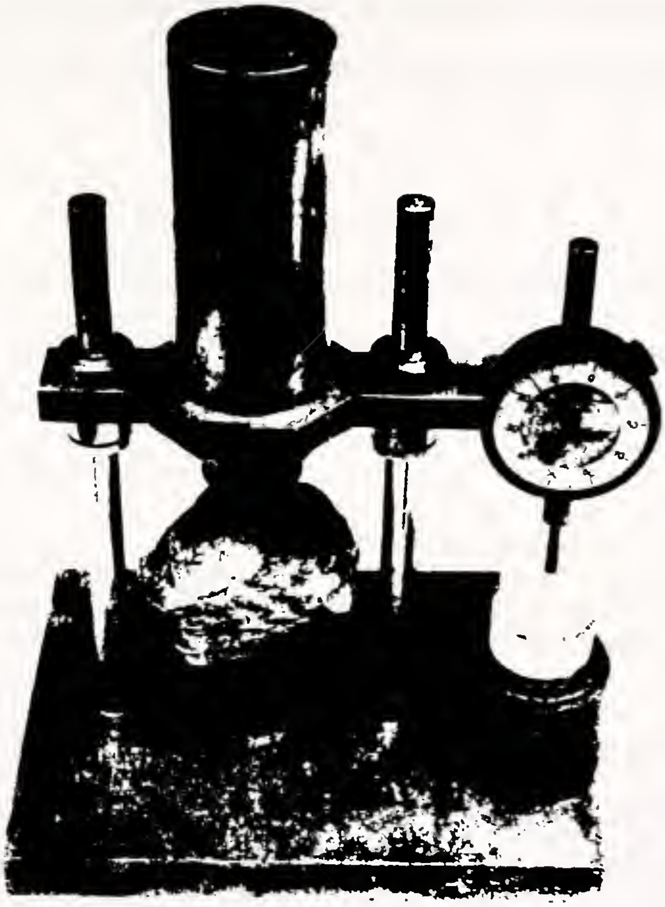


FIGURE 35 POINT LOAD STRENGTH APPARATUS

from the load gage of the compression machine and from the dial gage of the PLS apparatus.

All of the shale pieces tested eventually broke completely. However, to account for the successive failure displayed by some samples, the failure load was taken as the maximum load that had occurred any time during the loading process. The deformation corresponding to the maximum load value was either obtained directly or estimated from the recorded data.

5.2.6 Results and Discussion

The results of point load strength (PLS) tests conducted by Bailey (1976) on eighteen different shales provided evidence that the PLS test was sensitive to small strength variations in shales. Bailey (1976) concluded by stating;

> "In summation, point load strength testing appears to be an adequate method of measuring the strength of shale pieces. This strength measure may prove to be correlatable with such embankment construction parameters as ease of excavation and degradation due to compaction."

The hypothesis adopted for this testing sequence was that the PLS test could provide a strength parameter indicative of the resistance of the material to degradation. The value commonly produced from the PLS test is the index P/D^2 where P is the failure load and D is the initial sample thickness. The results from the PLS and standard compaction-degradation tests are given in Table 9. The PLS values were rather similar and did not correspond to the order or range of Index of Crushing (IC) values observed for the three shales.

Table 9. Results of Point Load Strength and Degradation Tests

Shale	PLS Tests		Degradation
	Average Sample Thickness cm (in)	Index P/D^2 MPa (psi)	IC (%)
New Providence	4.57 (1.8)	0.35 (50.8)	37.6
Osgood	5.08 (2.0)	0.31 (44.7)	25.1
Palestine	3.56 (1.4)	0.30 (42.9)	51.8

Smaller samples may allow a greater delineation among shales. However, decreasing the sample size could introduce additional variation and increase the size effect. Also, a minimum sample thickness may exist where the zones of compressive stresses and the critical region of tensile stresses within the sample begin to influence each other. An overlapping of the zones could alter the failure mechanism and in essence change the principles of the PLS test. With these potential problems, the testing of larger samples was preferred. However, the P/D^2 values from the larger samples were judged to be insufficient for predicting shale degradability.

With the rejection of the P/D^2 values, another parameter that could be obtained from the PLS test was necessary. Since both the load and the distance between the conical points were monitored during the PLS testing, a term reflecting the work required for failure could be calculated.

The changing distance between the loading points was viewed as a form of deformation which represented a combination of the elastic and plastic strain of the shale and the penetration of the loading points due to localized crushing. Although the penetration may not be considered a true deformation, it reflects a material property of crushing resistance and therefore was included in the strength parameter.

A plot of the load-deformation relation proved to be essentially linear, and a work input parameter was then calculated from;

$$WI = \frac{1}{2} (P \times Def.)$$

where WI is the work input, P is the failure load, and Def. is the sample deformation as previously defined at that load. The results of the calculations for the work input parameter are compared with the shale degradation values in Table 10. The similar values of the work input parameter for the New Providence and Palestine shales did not accurately reflect the difference in degradation properties of these materials. Materials with very different failure loads and deformation characteristics may ultimately include equivalent areas under their load-deformation curves.

Observation of the load-deformation relationships for each shale indicated that the concept of the slope of the resulting plot,

Table 10. Comparison of Work input and Shale Degradation Values

Shale	Work Input N - m (in - lb)	IC (%)
New Providence	0.659 (5.83)	37.6
Osgood	0.356 (3.15)	25.1
Palestine	0.664 (5.87)	51.8

rather than the area under the curve, might provide a suitable parameter. Using the P/D^2 value as a stress term and Def./D as a strain term, a general form of a modulus $[(P/D^2)/(Def./D]$ was established as a material characteristic. A comparison of the Modulus and degradation values is given in Table 11.

The Modulus term matches both the order and range of degradation values. A regression analysis performed on the data produced an exponential curve with the equation below for the Modulus expressed in MPa.

$$\ln IC = -4.7691 \times 10^{-2} \text{ (Modulus)} + 4.10$$

The following equation is used for the Modulus expressed in psi.

$$\ln IC = -3.289 \times 10^{-4} \text{ (Modulus)} + 4.10$$

Table 11. Comparison of Modulus and Shale Degradation Values

Shale	Modulus MPa (psi)	IC (%)
New Providence	9.44 (1369)	37.6
Osgood	18.49 (2681)	25.1
Palestine	3.38 (490)	51.8

The above relationships had an R^2 value of 0.998. A plot of the expression given in Figure 36 provides a graphical solution for the relationship between Modulus and IC values.

Although the curve was based on limited data, predictions of the Index of Crushing (IC) from some limiting Modulus values seem reasonable. The upper bound of the IC as the Modulus approaches zero is appropriate for the set of conditions used in the standard compaction-degradation tests. The equation also gives reasonable lower levels of the IC for the higher Modulus values possible in stronger shales.

5.2.7 Summary

The point load strength (PLS) test appears to be useful as an indicator of aggregate strength for shales. The Modulus determined from this test provided the best correlation with degradation values

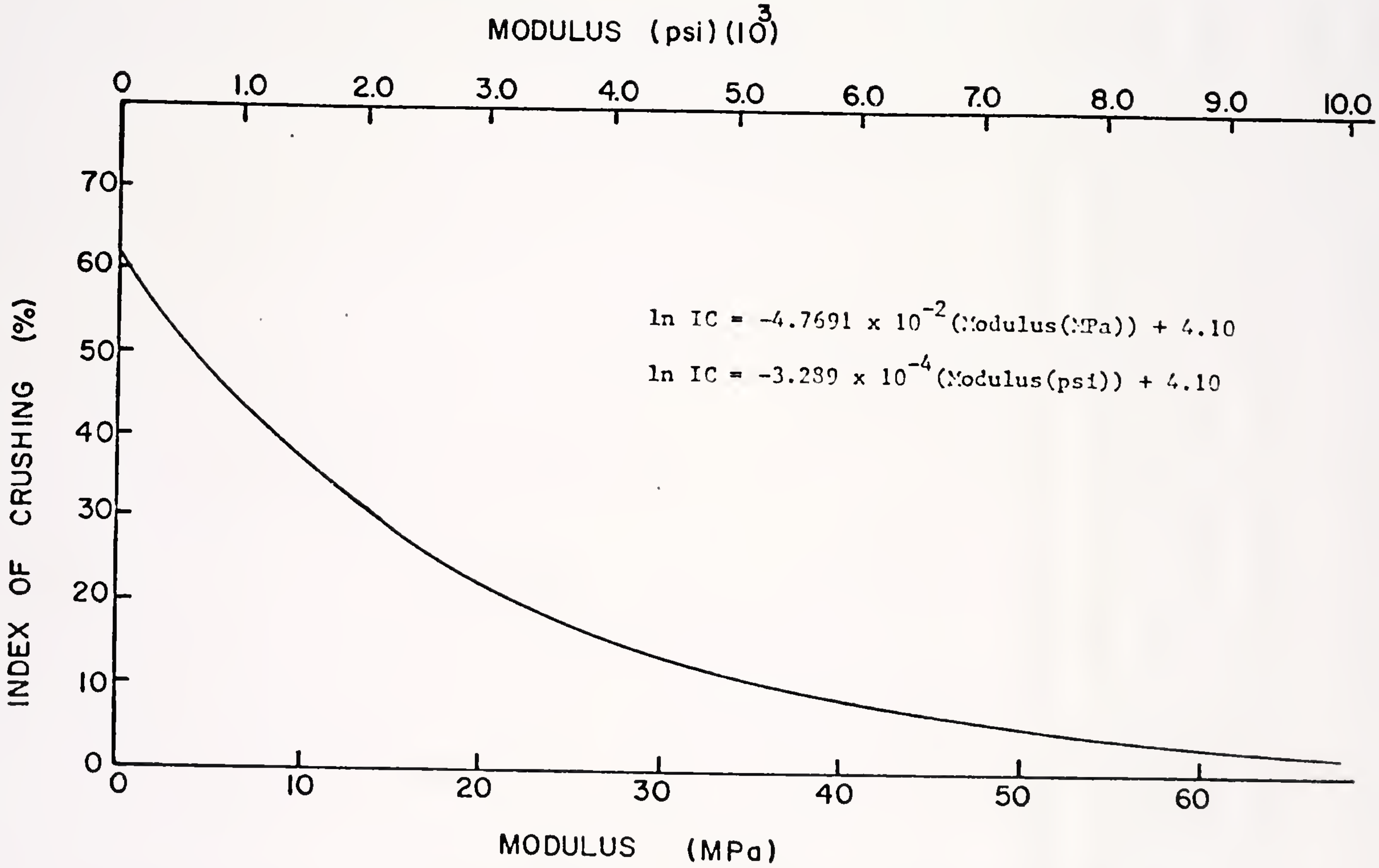


FIGURE 36 RELATIONSHIP BETWEEN MODULUS AND INDEX OF CRUSHING

and led to the formulation of a reasonable prediction equation for the Index of Crushing.

The PLS test is sufficiently simple and rapid to be performed conveniently in the field. Further experience in the use of this point load strength test will aid the prediction of shale degradability. Increased prediction capabilities could ultimately lead to the improvement of procedures and specifications for compacted shale embankments.

6. CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations are based on the results of the compaction and degradation studies for three Indiana shales.

6.1 Conclusions

(1) Both the impact and static compaction procedures developed in this study are suitable as a compaction - degradation test. However, past experience, simplicity, and equipment availability favor the impact test. Also the aggregate movement which occurred during impact compaction was considered to be more reflective of field compaction behavior than the more confined conditions in the static test. Therefore, the impact compaction procedure using the 790 $kN/m/m^3$ (16,500 $ft\text{-}lb/ft^3$) effort level is suggested for the standard compaction - degradation test.

(2) Limitations on degradation during impact or static compaction were not observed for the range of compactive efforts used in this study. The density curves for both impact and static compaction did show a tendency to be asymptotic at the higher levels of compactive effort. Although no actual limiting values were observed in this testing program, Bailey's (1976) conclusions that degradation and densification are self-limiting processes appear to be valid.

(3) The volume of voids of the compacted samples was nearly identical for a given effort level. However, the different final gradations for

each shale indicated that the distribution of pore sizes making up the total void volume was different for each shale. Therefore, a measure of the final gradation could serve as an indicator of the pore size distribution. Also a conjugate distribution of solid sizes could be readily obtained from the aggregate size distribution.

(4) The measurement of compactive work in the static compaction tests resulted in a linear relationship between density and work. In the relationship, limiting values of compactive work and density are reached simultaneously. The results from the static tests indicate that the compactive work term is a more realistic expression of compactive effort than the nominal compactive energy terms currently used.

(5) The analysis of initial gradation indicates that for a given compactive effort, the degradation decreases and the density increases as the initial sample gradation becomes more well-graded (increasing coefficient of uniformity D_{60}/D_{10}).

(6) The effect of maximum aggregate size was analyzed by using initial gradations with the same exponential cumulative distribution curves but different maximum sizes. The results showed that degradation increased as the maximum aggregate size increased. Also, the range in the Index of Crushing values for the smallest size gradations was less than for the largest size gradations. The greater distinction among the degradation of the shales observed for the larger maximum sizes emphasizes the importance of using samples with aggregate sizes as large as possible for laboratory testing.

The effect of the maximum size on the density was not well established. The differences in the densities for the three gradations

were small and demonstrated no general trend. A more pronounced effect on density may have been observed at a higher compactive effort.

(7) The addition of water to the shales led to the increased degradation for a given compactive effort through slaking and material strength reduction. The extent of the moisture effect on a shale will depend on the physical properties of the shale such as the magnitude and distribution of porosity, the mineralogical composition, and the durability. Both degradation and density will increase with increasing moisture content up to a limiting value. Beyond this point, additional moisture leads to entrapped air voids. The occluded air voids cushion the aggregates leading to reduced degradation and prevention of further densification.

(8) The standard impact test may be used successfully to classify the degradability of shales. The partial A gradation (Section 5.1.3) is recommended to simplify the test and to place a greater emphasis on the degradation of the larger aggregate sizes. Modifications in the standard impact test as a degradability index test may become necessary as more experience is gained.

(9) Results from the point load strength tests indicate that the commonly used P/D^2 index is not a sufficient indicator of shale degradability. However, the secant modulus from this study provided a good correlation with degradation values and led to the formation of a reasonable prediction equation for the Index of Crushing.

The point load test is sufficiently simple and rapid to be performed conveniently in the field. Despite the problems involved in testing shales, increased experience and prediction capabilities from

the point load test could ultimately lead to the improvement of procedures and specifications for compacted shale embankments.

In final summation, the suggested standard impact compaction - degradation test developed in this study meets the guidelines established in the testing program and may be successfully applied to the study of shale compaction and degradation behavior. The effects of initial gradation and maximum aggregate size on shale degradation are similar to the effects of these variables on highway aggregates. In this light, previous experience in the characteristics of highway mineral aggregates may be used as qualitative guidelines for understanding shale degradation behavior. The effect of moisture on shale is complex and depends on many different variables. In the laboratory, the addition of moisture does increase the degradation and density of the laboratory samples. In the field, there are problems of large aggregates, inadequate mixing, and inadequate curing. These limit the effectiveness of moisture in field compaction, although moisture addition in general is helpful. Finally, the ability to classify shale degradability from the standard impact test or the point load strength test could ultimately lead to improved and more efficient design and construction procedures for building compacted shale embankments.

6.2 Recommendations

The following recommendations are based on the conclusions of this study.

(1) Establish experience in the use of the suggested standard impact compaction - degradation test and increase the present knowledge of shale behavior by incorporating the test into a routine shale testing program.

(2) Establish correlations between the laboratory test results and observations of shale behavior during construction and in-service conditions.

(3) Improve degradability prediction capabilities by correlating the results from the suggested standard impact compaction - degradation test and point load strength test with shale degradation during field compaction. Also, evaluate the relationship presented in this study between degradation and the point load test as information from more shale types becomes available.

(4) Continue investigations of the size effect in point load testing and establish a correction procedure.

(5) Evaluate the relationship between the pore size distribution, solid size distribution, and degradation values of compacted shale samples.

BIBLIOGRAPHY

BIBLIOGRAPHY

Abeyesekera, R. A., (1978), "Stress-Deformation and Strength Characteristics of a Compacted Shale," Ph.D. Thesis and Joint Highway Research Project Report No. 77-24, Purdue University, West Lafayette, Indiana, May, 346 pages.

American Geological Institute, (1976), Dictionary of Geological Terms, Anchor Books, Garden City, New York, p. 394.

American Society for Testing and Materials, "Moisture-Density Relations of Soils Using 5.5-Lb. Rammer and 12-In. Drop," 1971 Book of ASTM Standards, Part II, Test Method D 698-70, pp. 332-337.

American Society for Testing and Materials, "Moisture-Density Relations of Soils Using 10-Lb. Rammer and 18-In. Drop," 1971 Book of ASTM Standards, Part II, Test Method D 1557-70, pp. 498-503.

American Society for Testing and Materials, "Classification of Soils for Engineering Purposes," 1971 Book of ASTM Standards, Part II, Test Method D 2487-69, pp. 796-802.

American Society for Testing and Materials, "Unconfined Compressive Strength of Intact Rock Core Specimens," 1971 Book of ASTM Standards, Part II, Test Method D 2938-71, pp. 935-936.

Andrews, Chris, (1979), Geologist, Indiana State Highway Commission, Division of Materials and Tests, Indianapolis, Indiana, Personal Communication.

Aughenbaugh, N. B., Johnson, R. B., and Yoder, F. J., (1963), Degradation of Base Course Aggregates During Compaction, School of Civil Engineering, Purdue University, West Lafayette, Indiana, May, 191 pages.

Austin, George S., (1975), "Clay and Shale Resources of Indiana," Indiana Geological Survey, Department of Natural Resources, Bulletin 42-L, 40 pages.

Bailey, M. J., (1976), "Degradation and Other Parameters Related to the Use of Shale in Compacted Embankments," MSCE Thesis and Joint Highway Research Project Report No. 76-23, Purdue University, West Lafayette, Indiana, August, 208 pages.

Bieniawski, Z. T., (1975), "The Point Load Test in Geotechnical Practice," Engineering Geology, Vol. 9, No. 1, pp. 1-11.

British Standards Institution, (1960), "10 Percent Fines Aggregate Crushing Test," BS 812, Pt. 3, Sect. 8, London, pp. 8-9.

Broch, E., and Franklin, J. A., (1972), "The Point Load Strength Test," International Journal of Rock Mechanics and Mining Sciences, Vol. 9, No. 6, November, pp. 669-697.

Brook, Norman, (1977), "A Method of Overcoming Both Shape and Size Effects in Point Load Testing," Proceedings of Rock Engineering Conference, University of Newcastle Upon Tyne, England, April, pp. 53-70.

Butler, P. E., (1976), "Utilization of Coal Mine Refuse in Highway Embankment Construction," Transactions of the Society of Mining Engineers of AIME, Vol. 260, No. 2, June, pp. 134-141.

Chapman, D. R., (1975), "Shale Classification Tests and Systems: A Comparative Study," MSCE Thesis and Joint Highway Research Project Report No. 75-11, Purdue University, West Lafayette, Indiana, June, 90 pages.

Chapman, D. R., and Wood, L. E., (1974), "A Questionnaire-Aided Assessment of the State of the Art for Compacted Shale Embankments," Joint Highway Research Project Report No. 74-19, Purdue University, West Lafayette, Indiana, December, 29 pages.

Chenevert, M. E., (1970), "Shale Alteration by Water Adsorption," Journal of Petroleum Technology, September, pp. 1141-1148.

Deo, P., (1972), "Shales as Embankment Materials," Ph.D. Thesis and Joint Highway Research Project Report No. 45, Purdue University, West Lafayette, Indiana, December, 202 pages.

Diamond, Sidney, (1978), Professor, Purdue University, West Lafayette, Indiana, Personal Communication.

Franklin, J. A., (1979), President, Franklin Trow Associates, Rexdale, Ontario, Canada, Personal Communication.

Garcia-Benochea, I., (1978), "The Relation Between Permeability and Pore Size Distribution of Compacted Clayey Silts," MSCE Thesis and Joint Highway Research Project Report No. 78-4, Purdue University, West Lafayette, Indiana, April, 179 pages.

Gaudette Jr., N. G., (1960), "Applications of the Kneading Compactor and Hveem Stabilometer to Bituminous Concrete Design in Indiana," Joint Highway Research Project Report No. 23, Purdue University, West Lafayette, Indiana, December, 238 pages.

Guidicini, G., Nieble, C. M., and Cornides, A. T., (1973), "Analysis of Point Load Test as a Method for Preliminary Geotechnical Classification of Rocks," Bulletin of the International Association of Engineering Geology, No. 7, June, pp. 37-52.

Harr, M. E., (1977), Mechanics of Particulate Media, McGraw-Hill International Book Company, New York, p. 9.

Harrison, J. L., and Murray, H. H., (1964), "Clays and Shales of Indiana," Indiana Geological Survey, Department of Conservation, Bulletin 31, pp. 25-32.

Hilf, J. W., (1975), "Compacted Fill," Foundation Engineering Handbook, Edited by H. F. Winterkorn and H. Y. Fang, Van Nostrand Reinhold Co., New York, pp. 244-311.

Hiramatsu, Y., and Oka, Y., (1966), "Determination of the Tensile Strength of Rock by a Compressional Test of an Irregular Test Piece," International Journal of Rock Mechanics and Mining Sciences, Vol. 3, No. 2, May, pp. 89-99.

Hobbs, D. W., (1964), "A Simple Method for Assessing the Uniaxial Compressive Strength of Rock," International Journal of Rock Mechanics and Mining Sciences, Vol. 1, No. 1, January, pp. 5-15.

Hodek, R. J., (1972), "Mechanism for the Compaction and Response of Kaolinite," Ph.D. Thesis and Joint Highway Research Project Report No. 72-35, Purdue University, West Lafayette, Indiana, December, 269 pages.

Hodek, R. J., and Lovell, C. W., (1978), "Soil Aggregates and Their Control Over Soil Compaction and Swelling," Presented at January 15-19 Meeting of TRB, Publication Pending.

Ingram, R. L., (1953), "Fissility of Mudrocks," Bulletin, Geological Society of America, Vol. 64, August, pp. 869-878.

International Society for Rock Mechanics, (1972), "Suggested Method for Determining the Point Load Strength Index," Commission on Standardization of Laboratory and Field Tests, Document No. 1, Part 2, Unpublished Final Draft, October, 5 pages.

Kinter, Earl B., and Diamond, Sidney, (1956), "A New Method for Preparation and Treatment of Oriented-Aggregate Specimens of Soil Clays for X-ray Diffraction Analysis," Soil Science, Vol. 81, No. 2, February, pp. 111-120.

Krynine, Dimitri P., and Judd, William R., (1957), Principles of Engineering Geology and Geotechnics, McGraw-Hill Book Company, New York, p. 53.

Lees, G., and Kennedy, C. K., (1975), "Quality, Shape, and Degradation of Aggregates," Quarterly Journal of Engineering Geology, Vol. 8, No. 3, pp. 193-209.

Loubser, M. M., (1967), "Shale in Road Foundations," Fourth Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Cape Town, South Africa, December, pp. 129-134.

Mead, W. J., (1936), "Engineering Geology of Damsites," Transactions of the 2nd International Congress on Large Dams, Washington, D.C., Vol. 4, p. 183.

Moriwaki, Y., (1975), "Causes of Slaking in Argillaceous Materials," Ph.D. Thesis, University of California at Berkeley, January, 291 pages.

Parker, J., (1970), "Temperature and Humidity Affect Strength of Rock Structures at White Pine," Transactions of the Society of Mining Engineers of AIME, Vol. 247, No. 2, June, pp. 142-144.

Pettijohn, F. J., (1957), Sedimentary Rocks, Harper & Row Publishers, New York, pp. 340-373.

Ramana Sastry, M. V. B., (1977), "Influence of Aggregate Gradings on the Degradation of Aggregates," Fifth Southeast Asian Conference on Soil Engineering, Bangkok, Thailand, July, pp. 361-375.

Reichmuth, D. R., (1968), "Point Load Testing of Brittle Materials to Determine Tensile Strength and Relative Brittleness," Proceedings of the Ninth Symposium on Rock Mechanics, Society of Mining Engineers of the AIME, pp. 134-159.

Shergold, F. A., and Hosking, J. R., (1959), "A New Method of Evaluating the Strength of Roadstone," Roads and Road Construction, Vol. 37, No. 438, June, p. 164.

Surendra, M., (1979), Graduate Research Assistant, Purdue University, West Lafayette, Indiana, Personal Communication.

Terzaghi, Karl, and Peck, Ralph B., (1967), Soil Mechanics in Engineering Practice, John Wiley and Sons, New York, p. 146.

Tschebotarioff, Gregory P., (1973), Foundations, Retaining, and Earth Structures, McGraw-Hill Book Company, New York, p. 102.

Underwood, L. B., (1967), "Classification and Identification of Shales," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 93, No. SM6, November, pp. 97-116.

U.S. Army Engineer Waterways Experiment Station, (1975), "Design and Construction of Compacted Shale Embankments--Vol. 1: Survey of Problem Areas and Current Practices," Technical Report FHWA-RD-75-61, August, 292 pages.

Van Eeckhout, E. M., (1976), "The Mechanisms of Strength Reduction Due to Moisture in Coal Mine Shales," International Journal of Rock Mechanics and Mining Sciences, Vol. 13, No. 2, February, pp. 61-67.

Van Zyl, D., (1977), "Storage, Retrieval, and Statistical Analysis of Indiana Shale Data," Joint Highway Research Project Report No. 77-11, Purdue University, West Lafayette, Indiana, July, 140 pages.

Witsman, Gary R., (1979), "The Effect of Compaction Prestress on Compacted Shale Compressibility," MSCE Thesis and Joint Highway Research Project Report No. 79-16, September, 181 pages.

Wood, L. E., Lovell, C. W., and Deo, P., (1973), "Building Embankments with Shales," Joint Highway Research Project Report No. 16, Purdue University, West Lafayette, Indiana, August, 32 pages.

Wood, L. E., Sisiliano, W. J., and Lovell, C. W., (1978), "Guidelines for Compated Shale Embankments," Highway Focus, Vol. 10, No. 2, May, pp. 22-36.

APPENDICES

This copy of this Report does not contain the several Appendices. They are as follows:

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